Geotechnics
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San Diego
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GEOTECHNICAL INVESTIGATION
GAS TURBINE POWER PLANT
NILAND, CALIFORNIA

prepared for

Imperial Irrigation District
Post Office Box 937
Imperial, CA 92251

by

GEOTECHNICS INCORPORATED
Project No. 0554-075-00
Document No. 06-0015

February 10, 2006

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Attention: Mr. Baltazar Aguilera

SUBJECT: GEOTECHNICAL INVESTIGATION
Gas Turbine Power Plant
Niland, California

Dear Mr. Aguilera:

In accordance with your request, we have completed a geotechnical investigation for the proposed Gas Turbine Power Plant in Niland, California. Specific conclusions regarding site conditions and recommendations for foundations and earthwork are presented in the attached report.

We appreciate this opportunity to provide professional services. If you have any questions or comments regarding this report or the services provided, please do not hesitate to contact us.

GEOTECHNICS INCORPORATED

Robert A. Torres, P.E.
Principal Engineer

Distribution: (4) Addressee, Mr. Baltazar Aguilera

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GAS TURBINE POWER PLANT
NILAND, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Gas Turbine Power Plant in Niland, California. The purpose of this investigation was to characterize the pertinent geotechnical conditions at the site, and provide recommendations for the geotechnical aspects of the proposed plant. The conclusions presented in this report are based on field exploration, laboratory testing, engineering analysis, and our previous experience with similar soils and geologic conditions.

2.0 SCOPE OF SERVICES

This investigation was conducted in general accordance with the provisions of our Proposal No. 05-373 (Geotechnics, 2005). In order to evaluate geotechnical impacts to the proposed development, and to provide recommendations for design and construction of the proposed power plant, the following services were provided.

- A reconnaissance of the surface characteristics of the site. This included a literature review of available maps, reports, and aerial stereoscopic photographs of the site and adjacent properties. Pertinent references are provided in Appendix A.

- A subsurface exploration of the site including 12 hollow-stem auger borings and 6 cone penetrometer soundings at the locations previously determined by the Imperial Irrigation District. Selected samples of the materials encountered in the explorations were collected for laboratory analysis. Logs of the explorations are presented in the figures of Appendix B.

- In-situ percolation testing of the surficial soils within the proposed storm water detention basins. Percolation tests were conducted at three locations in general accordance with the *Imperial County Uniform Policy and Method for Soils Evaluation, Testing and Reporting.* The percolation test results are summarized in Appendix C.

- In-situ earth and thermal resistivity testing at two locations within each of the areas for the Turbine Generator, GSU and Switchyard. The soil resistivity testing was conducted by M. J. Schiff & Associates using the four point method (IEEE Standard 81 and 442, respectively), and is presented in Appendix D.

- Laboratory testing of selected samples collected during the subsurface exploration. Testing was intended to characterize and assess the pertinent engineering properties of the on site soils. Laboratory testing included gradation, hydrometer, Atterberg Limits, moisture content, dry density, expansion, corrosion and shear strength. The laboratory test results are summarized in Appendix E.

- Assessment of general seismic conditions and geologic hazards affecting the site vicinity, and their likely impact on the project. Our liquefaction analysis is presented in Appendix F.

- Engineering and geologic analysis of the field and laboratory data in order to develop recommendations for earthwork construction, site preparation, remedial grading recommendations, mitigation of expansive and compressible soil conditions beneath pads, fill and backfill placement, and foundation recommendations for the proposed structures. Alternative foundations were evaluated including spread footings, mat foundations and pile foundations. Our deep foundation analyses are presented in Appendix G.

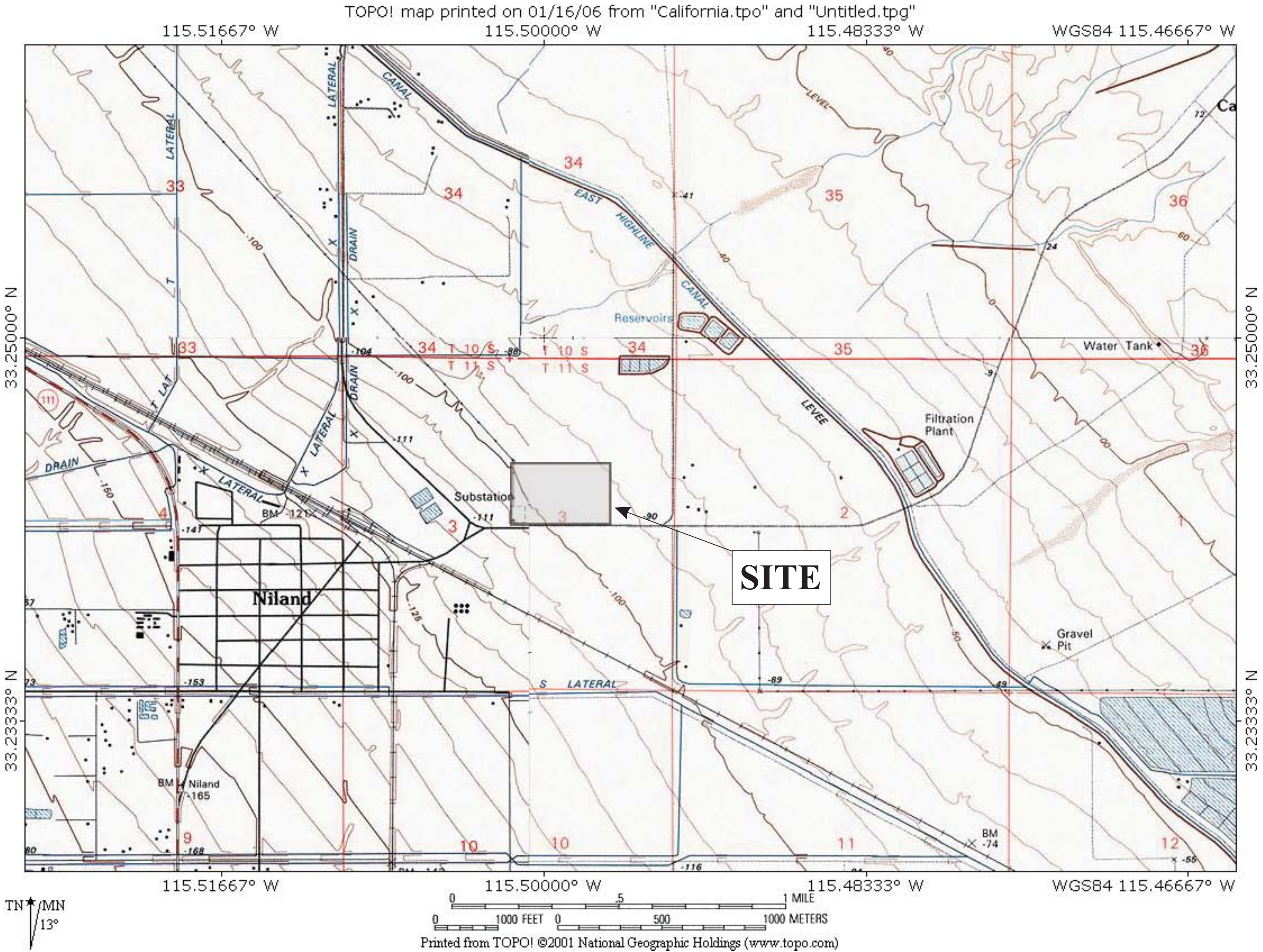
- Preparation of this report summarizing our findings, conclusions and recommendations.

3.0 SITE DESCRIPTION

The site is located about ½ mile east of the City of Niland, California, as shown on the Site Location Map, Figure 1. Beal Road provides access to the property, and forms the southern boundary. The southwest corner of the property contains an existing electric substation. The site is bordered by undeveloped land to the west, north and east. The property is rectangular in shape, and is approximately 1,000 feet long and 1,500 feet wide. The site and surrounding areas slope gently to the southwest (toward the Salton Sea). According to the program TOPO!, the site is located between approximately 90 and 100 feet below mean sea level (Wildflower, 1997). The layout of the property is shown on the Exploration Plan, Figure 2.

4.0 PROPOSED DEVELOPMENT

The proposed development is anticipated to include the construction of two quick-start General Electric LM6000 gas combustion turbine generators capable of producing a total of 90 megawatts of electricity during peak power demand periods. The generators are scheduled to be incorporated into the Imperial Irrigation District's power supply network in May of 2008.



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SITE LOCATION MAP

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FIGURE 1

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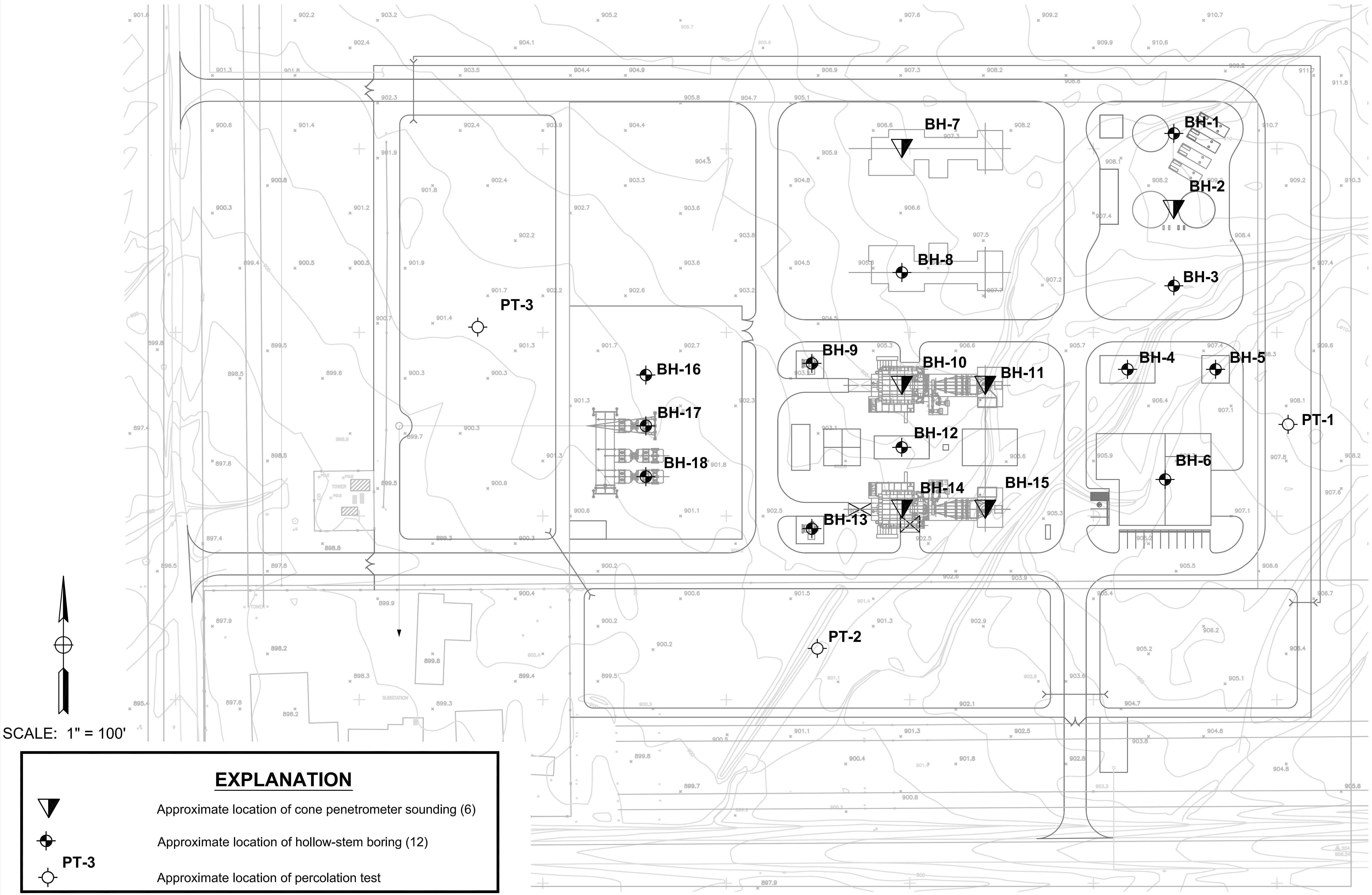

EXPLANATION

◣	Approximate location of cone penetrometer sounding (6)
⊕	Approximate location of hollow-stem boring (12)
PT-3 ○	Approximate location of percolation test

Reference: C1_2 Rev A Boring Location Plan.pdf, provided by POWER Engineers

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FIGURE 2

EXPLORATION PLAN

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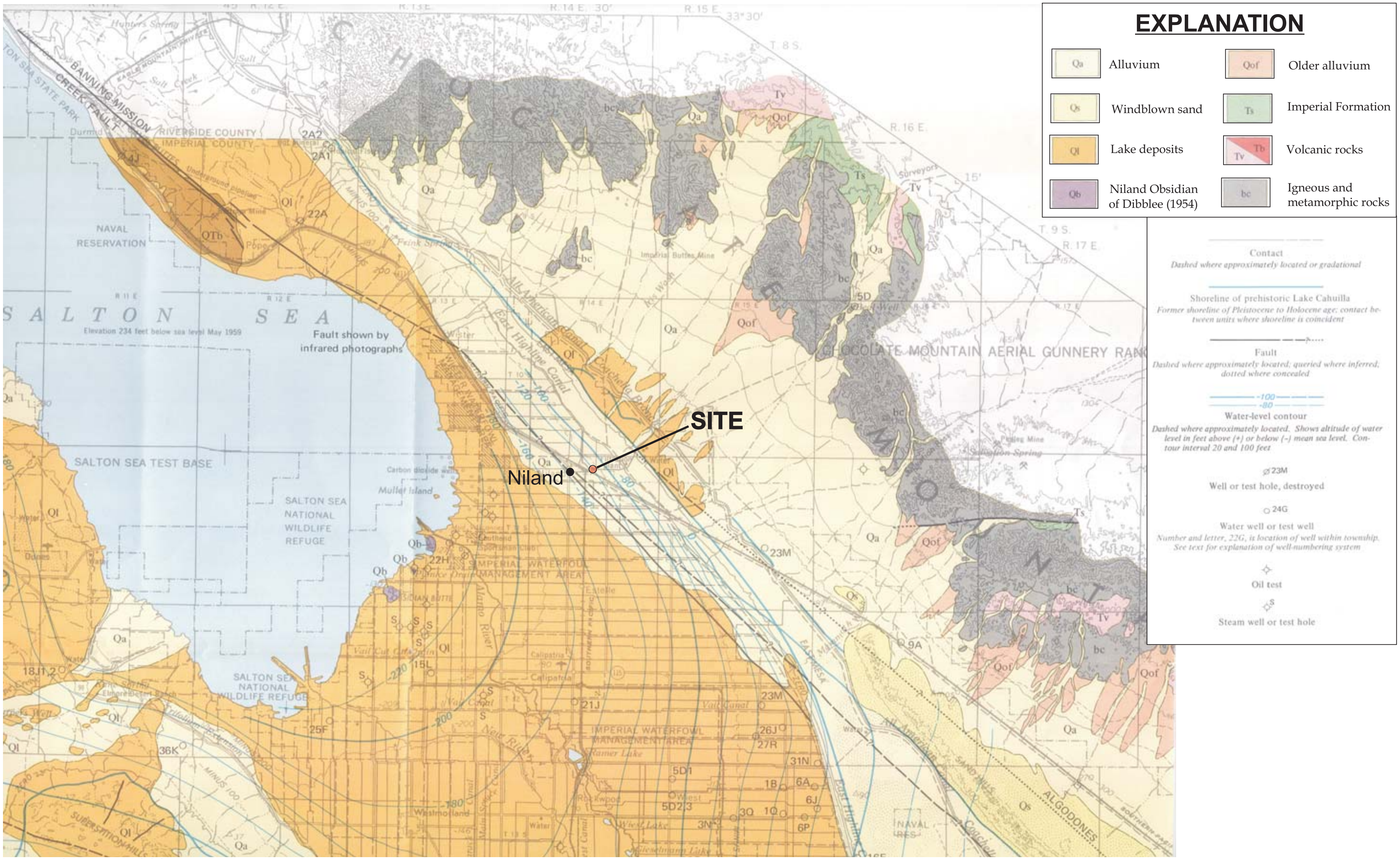
In addition to the generators, development will include construction of a one-story office building with a control room and warehouse, a variety of electrical equipment pads, two water tanks (35 feet in diameter and 32 feet tall), several storm water detention basins, and various paved driveways and parking areas. We anticipate that the generators and water tanks will be supported on mat foundations or pile caps (maximum equipment loads are on the order of 460 kips). The approximate layout of the proposed development is also shown on the Exploration Plan, Figure 2.

5.0 GEOLOGY AND SUBSURFACE CONDITIONS

The site is situated within the south-central portion of the Salton Trough, a topographic and structural depression bound to the north by the Coachella Valley and to the south by the Gulf of California. The Salton Trough is a region of transition from the extensional tectonics of the East Pacific Rise to the transform tectonic environment of the San Andreas system. Late Cenozoic extension associated with the opening of the Gulf of California formed this deep topographic and structural depression (Elders, 1979). The marine water of the gulf was cut off by growth of the Colorado River delta, resulting in the closed basin present today.

The Salton Trough is an actively growing rift valley in which sedimentation has almost kept pace with tectonism (Elders, 1979). As rifting occurred, the Colorado River delta filled the trough, and conditions gradually changed from marine, to deltaic, to subaerial river and lake deposits. Today, the Mesozoic-age crystalline basement rocks of the trough are covered by about 15,000 feet of Cenozoic marine and nonmarine sedimentary deposits. During the Late Pleistocene and Holocene, the basin was periodically inundated by floodwaters of the Colorado River to form lakes. Lake Cahuilla was formed during the last 1,000 years and evidence of its shoreline are still present around the Imperial Valley. The latest flooding, in 1905, created the present-day Salton Sea (Sharp, 1982).

The approximate locations of the 12 exploratory borings and 6 cone penetrometer soundings conducted for this investigation are shown on the Exploration Plan, Figure 2. The general geologic conditions in the vicinity of the site are depicted on the Regional Geologic Map, Figure 3. Logs describing the subsurface conditions encountered in the explorations are presented in Appendix B. The geotechnical characteristics of the materials at the site are discussed below.



Modified From: USGS Loeltz et al 1975, Plate 1

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REGIONAL GEOLOGIC MAP

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FIGURE 3

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5.1 Lacustrine Deposits

The subject site is underlain by lacustrine deposits associated with the ancient lakes which occupied the area. In general, the lacustrine deposits encountered in our subsurface exploration include thick sequences of lean to fat clay (Unified Soil Classification Symbol CL to CH) with thin interbedded lenses of sandy silt (ML). The lacustrine deposits were generally dry to moist, moderately to highly expansive, and hard in consistency. The average dry density was 108 lb/ft^3, with an average moisture content of 19 percent. Our observations suggest that the lacustrine deposits at the subject site may be older and more indurated compared to other areas in Imperial Valley, probably due to repeated cycles of desiccation.

The cone penetrometer tip resistance in these deposits generally varied from 45 to 65 TSF. Shear wave velocity measurements at the location of the turbine generator suggest that the site has an average shear wave velocity (v_s) of approximately 650 ft/s, which indicates a UBC Seismic Soil Profile S_D (see Appendix B). This corresponds to a dynamic shear modulus (G_{max}) of about 1,150 TSF, and a dynamic constrained modulus (E_s) of about 3,680 TSF. Note that these are upper bound estimates associated with small strains in hard, unsaturated clay. These clays will soften substantially with wetting and swelling, or strain in general. Three percolation tests were conducted in the lacustrine deposits as described in Appendix C. The tests suggest that the percolation rate of the lacustrine deposits ranges from 0 to ¼ gallon per square foot per day. The field resistivity test results are described in Appendix D.

5.2 Alluvium

A thin cover of alluvium mantles the lacustrine deposits at the site. The alluvium is generally 1 to 2 feet thick across the site. The alluvium typically consists of fine to coarse grained, well graded sand with silt and gravel (SW-SM). This material was dry and very loose in consistency. The alluvium is considered to be compressible.

5.3 Groundwater

No groundwater was observed within 91 feet of the surface in any of the 18 explorations conducted at the site. However, it should be noted that perched groundwater could develop in the future due to changes in site drainage, irrigation, or antecedent rainfall. Groundwater contours shown on the Regional Geologic Map, Figure 3, suggest that groundwater may have been closer to the surface when that map was published (Loeltz et al., 1975).

6.0 TECTONIC FRAMEWORK

The Salton Trough may have originally formed as a major half-graben during the regional crustal extension that took place in much of western North America in the Miocene (Frost et al., 1997). The Salton Trough is now a zone of transition between the ocean-floor spreading regime of the East Pacific Rise in the Gulf of California and the transform tectonic environment of the San Andreas fault system (Elders, 1979). Relative plate motion between the North American plate and Pacific plate is thought to be transferred to the San Andreas Fault near the south end of the Salton Sea (Sharp, 1982; Sylvester, 1976). Geophysical studies indicate the presence of a steep gravity gradient across the San Andreas fault along the eastern edge of the Trough (Biehler, et al., 1964). This gravity gradient indicates the northwest trending San Andreas fault is the principal structural boundary between the Salton Trough and the North American plate (Sylvester, 1976).

The Orocopia and Chocolate Mountains represent the broken edges of the North American plate along the eastern margin of the Salton Trough and are included in the southern Basin and Range physiographic province (Frost et al., 1997). The eastern edge of the Pacific plate is composed of intermediate composition granitic rocks of the Peninsular Ranges physiographic province. The eastern edge of the plate has been offset along multiple strands of the San Andreas system. The Salton Trough occupies the structurally weak zone between the strong, solid edges of the Pacific and North American plates. A zone of high seismicity connects the San Andreas fault north of the Salton Sea and the Imperial fault south of Brawley. This structurally low area (the Brawley Seismic Zone) may be the result of tensional or releasing step between the San Andreas and Imperial faults.

Potential seismogenic sources near the site include the San Andreas Fault, the Brawley Seismic Zone, the Imperial Fault, and the Elmore Ranch fault zone. Due to its proximity to the site, we have also included a discussion of the Sand Hills – Algodones fault zone although it is not a recognized seismogenic source. Each of these faults is described in greater detail below.

6.1 San Andreas Fault

The Coachella Valley segment of the San Andreas Fault is located approximately 23 kilometers north of the site. The San Andreas Fault has not been mapped south of the Salton Sea. While a linear extension of the fault may exist under the Salton Sea or in the northern Imperial Valley, there is no geologic or geophysical evidence to support it (Sharp, 1982). The California Division of Mines and Geology estimates a slip rate of 25 mm/year and a maximum moment magnitude of 7.7 for this segment of the San Andreas Fault.

Although the San Andreas Fault has produced a few moderate-sized earthquakes in historic times, no large earthquake has been documented on the San Andreas system south of San Bernadino (Hutton and Others, 1991). This 'locked' southernmost section of the fault also lacks microseismicity, and stands in sharp contrast to the northern sections of the fault which have ruptured with the largest historical earthquakes in California.

6.2 Brawley Seismic Zone

The Brawley Seismic Zone is located approximately 13 kilometers west of the site. The Brawley Seismic Zone is characterized by earthquake swarms generally less than magnitude 3 or 4. The California Division of Mines and Geology estimates a slip rate of 25 mm/year and a maximum moment magnitude of 6.4 for the Brawley Seismic Zone. The Brawley Seismic Zone is believed to separate the San Andreas Fault to the northeast and the Imperial fault to the southwest. The Brawley Seismic Zone was first recognized after several earthquake swarms between 1973 and 1979. These events defined lineations transverse to the strike of the Imperial fault (Johnson and Hill, 1982). Two types of earthquake swarms appear to occur in the Brawley Seismic Zone. Swarms in the south end of the zone near the town of Brawley tend to occur in pairs, nucleating on the Imperial fault and propagating north into the Seismic Zone. Swarms in the northern part of the zone nucleate within the zone and do not occur in pairs (Hutton and Others, 1991; Johnson and Hill, 1982). These swarms appear to be triggered by creep events on the Imperial fault (Johnson and Hill, 1982).

6.3 Imperial Fault

The Imperial fault is located about 33 kilometers southeast of the site. The Brawley fault is the northeastern branch of the Imperial fault, and was generally unrecognized until surface rupture occurred in 1975 (Sharp, 1976). The Brawley fault ruptured with the southern portion of the Imperial fault in 1979, confirming the relationship between these segments.

Historical seismicity suggests that a major portion of the displacement observed on the Imperial Fault is being transferred to the San Andreas Fault to the northeast through the Brawley Seismic Zone (Hutton and Others, 1991). The Imperial fault has a similar strike as the Coachella segment of the San Andreas Fault. Most of the aftershocks following the 1979 earthquake on the Imperial fault occurred within the Brawley Seismic Zone (Sharp, 1982). The California Division of Mines and Geology estimates a slip rate of 20 mm/year and a maximum moment magnitude of 7.0 for the Imperial fault.

6.4 Elmore Ranch Fault Zone

The Elmore Ranch fault zone is located approximately 30 kilometers west of site. The fault zone is composed of six northeast-southwest trending parallel segments up to 12 kilometers in length each. These left-lateral faults are conjugate faults (cross-faults) between the Brawley Seismic Zone to the east and the San Jacinto fault zone to the west. The California Division of Mines and Geology estimates a combined slip rate of 1½ mm/year, and a maximum moment magnitude of 6.1 for the Elmore Ranch fault zone.

6.5 Sand Hills – Algodones Fault

Many published geologic maps, including the Geologic Map of California (Jennings, 1994), show several inferred fault traces near the site with a northwest-southeast trend, commonly known as the Sand Hills-Algodones fault. The existence of this fault is based on somewhat ambiguous data such as anomalous topography and lineaments on aerial photographs, groundwater barrier effects in test wells, and magnetic gradients and gravity patterns of the Upper Mesa area in southeastern Yuma, Arizona. Seismic-reflection and refraction profiles, which form the basis of this interpretation, were conducted near Yuma, Arizona, and indicate a very steeply dipping basement contact, which is a possible indicator of a fault (Mattick et al., 1973). These faults have been inferred northwestward on many regional maps because they conveniently line up with the strike of the southern San Andreas Fault. If the Sand Hills-Algodones faults do exist in the vicinity of the project site, they would not be considered active, as the youngest sedimentary rocks unaffected by the inferred fault are “almost certainly older than the latest Pleistocene” (Mattick et al., 1973).

7.0 GEOLOGIC HAZARDS

The subject site is located within one of the most seismically active areas in California. The primary geologic hazards at the site are associated with the potential for strong ground shaking. Other potential geologic hazards may include liquefaction, lateral spread, earthquake induced flooding, and volcanic eruption. Each of these hazards is discussed in greater detail below.

7.1 Surface Rupture

Surface rupture is the result of movement on an active fault reaching the surface. The site is located in close proximity to the inferred location of the Sand Hills – Algodones faults, which are considered potentially active. The site is not located within an Alquist-Priolo Earthquake Fault Zone, and no evidence of active faulting was found during our investigation. Consequently, surface rupture is not considered to be a substantial geologic hazard at the site.

7.2 Seismicity

The approximate centroid of the proposed improvements is located at latitude 33.2432° north and longitude 115.4993° west. The Fault Location Map, Figure 4, shows the locations of known active faults within a 100 km radius of the site. Table 1 summarizes the properties of these faults based on the program EQFAULT and supporting documentation (Blake, 2000).

In order to provide an estimate of the peak ground accelerations that structures founded at the site may experience in time, the program FRISKSP was used perform a probabilistic analysis of seismicity. The analysis was conducted using the characteristic earthquake distribution of Youngs and Coopersmith (1985). Based on the results of our probabilistic analysis, the *Upper Bound Earthquake* for the site, defined as the motion having a 10 percent probability of being exceeded in a 100 year period, is 0.60g. The *Design Basis Earthquake* is 0.52g (10 percent probability in 50 years). By comparison, the California Geological Survey website estimates that the *Design Basis Earthquake* for the site is 0.38g (CGS, 2003).

7.3 Liquefaction and Dynamic Settlement

Liquefaction is a process in which soil grains in a saturated sandy deposit lose contact due to ground shaking. The soil deposit temporarily behaves as a viscous fluid; pore pressures rise, and the strength of the deposit is greatly diminished. Liquefaction is often accompanied by sand boils, lateral spread, and post-liquefaction settlement as the pore pressure dissipates. Liquefiable soils typically consist of cohesionless sands and silts that are loose to medium dense, and saturated. Clayey soils do not liquefy because the soil skeleton is not supported by grain to grain contact, and is therefore not subject to densification by shaking.

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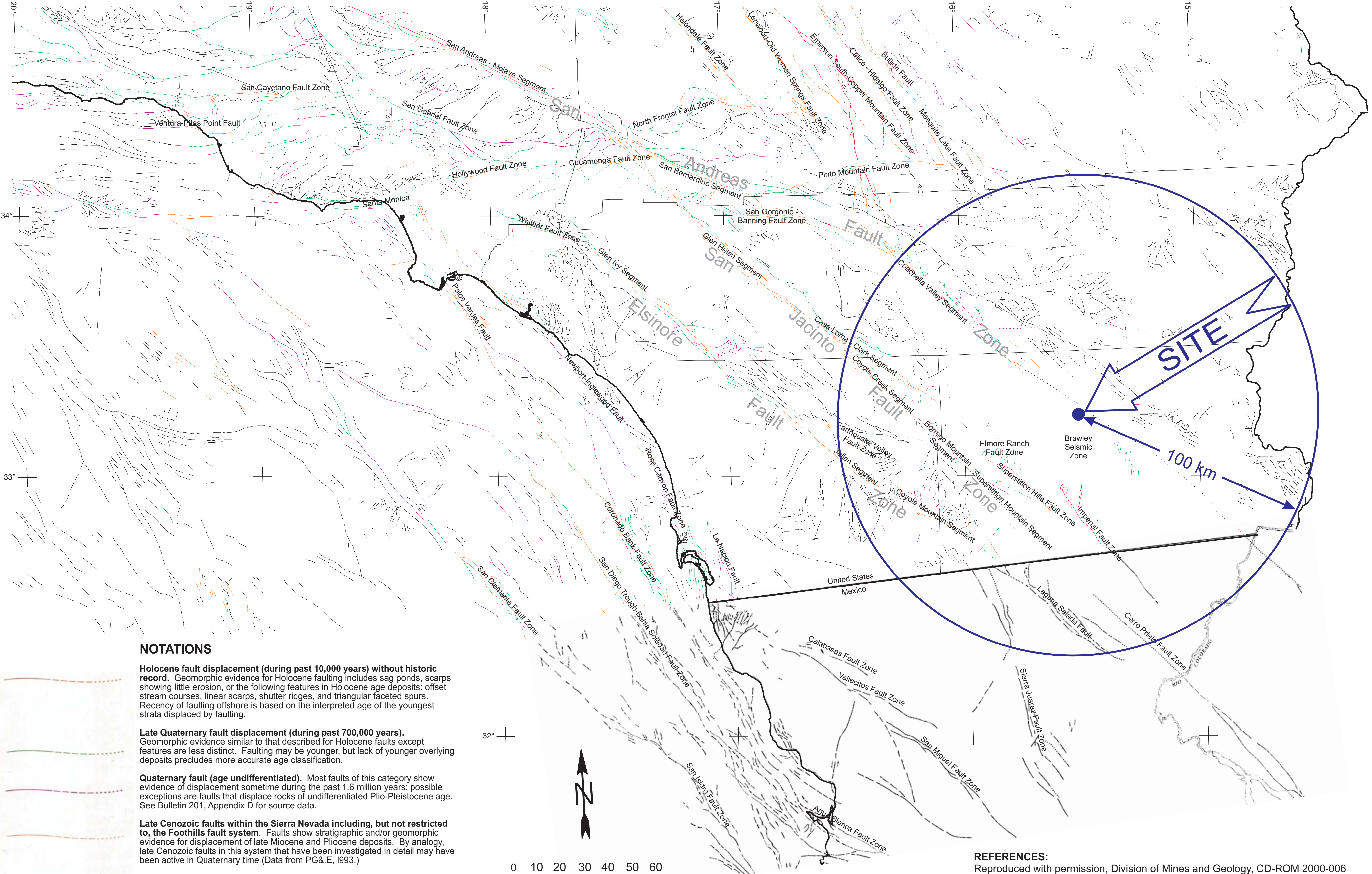
FIGURE 4

FAULT LOCATION MAP



NOTATIONS

Holocene fault displacement (during past 10,000 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults that displace rocks of undifferentiated Plio-Pleistocene age. See Bulletin 201, Appendix D for source data.

Late Cenozoic faults within the Sierra Nevada including, but not restricted to, the Foothills fault system. Faults show stratigraphic and/or geomorphic evidence for displacement of late Miocene and Pliocene deposits. By analogy, late Cenozoic faults in this system that have been investigated in detail may have been active in Quaternary time (Data from PG&E, l993.)

Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

REFERENCES:
Reproduced with permission, Division of Mines and Geology, CD-ROM 2000-006 (2000), Digital Database of faults from the Fault Activity Map of California and Adjacent Areas. IBID (1994), Selected Faults in Northern Baja California, Offshore, and the Adjacent Southern California Area.

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FAULT[1]	DISTANCE TO SITE [KM]	ESTIMATED PEAK GROUND ACCELERATION[2]	MAXIMUM EARTHQUAKE MAGNITUDE[3,5]	ESTIMATED FAULT AREA[4] [CM^2]	SHEAR MODULUS[4] [DYNE/CM^2]	ESTIMATED SLIP RATE[4] [MM/YEAR]
Brawley Seismic Zone	13	0.21	6.4	2.52E+12	3.30E+11	25.00
San Andreas - Whole M-1A	23	0.28	8.0	6.86E+13	3.30E+11	30.00
San Andreas - Sb-Coach. M-1B-2	23	0.25	7.7	2.43E+13	3.30E+11	27.00
San Andreas - Sb-Coach. M-2B	23	0.25	7.7	2.43E+13	3.30E+11	24.00
San Andreas - Coachella M-1C-5	23	0.20	7.2	1.15E+13	3.30E+11	25.00
Elmore Ranch (East)	30	0.11	6.1	1.11E+12	3.30E+11	1.50
Elmore Ranch (West)	31	0.11	6.1	1.32E+12	3.30E+11	1.50
Imperial (Model B)	33	0.13	7.0	7.75E+12	3.30E+11	20.00
Imperial (Model A)	34	0.13	7.0	7.92E+12	3.30E+11	20.00
Superstition Hills	38	0.09	6.6	3.89E+12	3.30E+11	4.00
Superstition Mountain	43	0.08	6.6	1.61E+12	3.30E+11	5.00
San Jacinto-Coyote Creek	49	0.08	6.8	6.15E+12	3.30E+11	4.00
San Jacinto - Borrego	51	0.06	6.6	3.48E+12	3.30E+11	4.00
San Jacinto-Anza	58	0.08	7.2	1.62E+13	3.30E+11	12.00
Laguna Salada	68	0.06	7.0	1.01E+13	3.30E+11	3.50
Elsinore (Coyote Mountain)	70	0.05	6.8	5.70E+12	3.30E+11	4.00
Elsinore (Julian)	86	0.05	7.1	1.13E+13	3.30E+11	5.00
Earthquake Valley	87	0.03	6.5	3.00E+12	3.30E+11	2.00
Cerro Prieto	93	0.04	7.1	1.16E+13	3.30E+11	20.00

1. Fault activity determined by Blake (2000), CDMG (1992), Wesnousky (1986), and Jennings (1994).
2. Median peak horizontal ground accelerations (in g's) from Sadigh (1997) for Soil Sites for the Maximum Earthquake Magnitude.
3. Moment magnitudes determined from CDMG (2003), Blake (2000), Wesnousky (1986) and Anderson (1984).
4. Estimated fault areas, shear moduli, and slip rates after fault data for EQFAULT and FRISKSP, Blake (2000).
5. The Maximum Earthquake Magnitude is the estimated median moment magnitude that appears capable of occuring given rupture of the entire estimated fault area.

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TABLE 1

It should be emphasized that no groundwater was observed in the 18 explorations conducted at the site, and that the underlying lacustrine deposits are primarily composed of hard clay. Consequently, the potential for liquefaction to occur is considered to be low. However, in order to provide an estimate of the potential for dynamic settlement at the site (which may occur on dry sands), liquefaction analysis was performed using the cone penetrometer data in general accordance with the referenced guidelines (SCEC, 1999). The results of the liquefaction analyses are presented in Figures F-1.1 through F-6.4 in Appendix F. The CPT method of liquefaction analysis is described in greater detail in Appendix F.

Several parameters are used to evaluate liquefaction and dynamic settlement. Liquefaction is not considered to be a hazard in clays. For our analysis, we assumed that soils with a Soil Behavior Type Index (Ic) greater than 2.6 were too clayey to liquefy or settle. Dense sands do not liquefy. For our analysis, sandy soils with a corrected CPT tip resistance $(q_{c1N})_{cs}$ greater than 160 were deemed too dense to liquefy (seismic settlement was included in our analysis for sands with a normalized tip resistance up to 200). The parameters Ic and $(q_{c1N})_{cs}$ are plotted as a function of depth in Figures F-1.1 through F-6.4.

Our analysis suggests that dynamic settlement may vary from about 1½ to 2 inches at the site. Much of this settlement is estimated to occur within the alluvium and surficial lacustrine deposits. If a 4 foot thick compacted fill mat is constructed beneath all site improvements, the total dynamic settlement would be reduced to approximately ½ to 1½ inches. According to state guidelines, a differential settlement equal to one-half of the anticipated total dynamic settlement may be conservatively assumed for structural design (SCEC, 1999). Consequently, we estimate that dynamic differential settlement across the length of the proposed structures will typically be less than ¾ inch at the site.

7.4 Landslides and Lateral Spreads

No evidence of active landslides was observed during our subsurface investigation. The site slopes very gently down from the northeast to the southwest, with approximately 10 feet of fall in 1,000 feet (a one percent gradient). Static landslides are not believed to present a significant hazard to the proposed development. However, the gradient may be large enough to produce a lateral spread, given locally continuous granular bedding, saturated conditions, and strong ground shaking. Such conditions were not observed on site. Consequently, lateral spread is not considered to be a significant geologic hazard.

7.5 Flooding

The site is situated within an active alluvial floodplain. Extensive gullies and channels were observed crossing the property and throughout the site vicinity, as shown on the Exploration Plan, Figure 2. Surface water flow at the site is likely the result of periodic intense, prolonged rainfall events. It is our understanding that site improvements will include construction of several detention basins to manage storm water runoff.

7.6 Tsunamis, Seiches, Earthquake Induced Flooding

The site is situated about 90 to 100 feet below sea level, as shown on the Site Location Map, Figure 1. This suggests that the potential may exist for inundation in the event of a tsunami within the Gulf of California. However, the configuration of the Gulf of California, and the higher ground surface elevation near Calexico, has historically provided relief from such events. There are no records which indicate that tsunamis have impacted the Imperial Valley in the last several hundred years. The distance between the subject site and the gulf most likely precludes damage due to seismically induced waves (tsunamis). The site is located more than 100 feet above the Salton Sea, and there are no other large bodies of water in close proximity. Consequently, the potential for seiches or earthquake induced flooding to adversely affect the site is also considered to be low.

7.7 Volcanic Hazards

Two north-south oriented tensional spreading centers have been identified in the Salton Trough based on geophysical surveys and recent volcanic activity (Kerr and Kidwell, 1991; Fuis and Kohler, 1984). One spreading center is located in the southern end of the Trough, approximately 30 kilometers south of the international border. The second spreading center is in the northern end of the Trough, and extends from the southern part of the Salton Sea to the City of Brawley. Volcanic activity associated with these spreading centers has reached the surface and formed the Cerro Prieto volcano in Baja California and the Salton Buttes just west of the site. The Salton Buttes consist of a group of five small extrusive volcanic domes.

The site is located more than 10 km east of extrusive rhyolite dome known as Obsidian Butte (a member of the Salton Buttes). The USGS includes the “Salton Buttes rhyolite center” among the Potential Areas of Volcanic Hazards (USGS Bulletin 1847). USGS studies suggest that a single eruption occurred about 16,000 years ago, with no subsequent activity.

According to the USGS, the most probable future potential hazard would be explosive and extrusive rhyolitic eruptions, or phreatic and phreatomagmatic eruptions (volcanic eruptions or explosions of steam and mud caused by the heating of groundwater). Since no recurrence interval can be estimated, the USGS has not quantified the potential hazard.

7.8 Subsidence

The site is subject to subsidence from regional tectonic processes as well as localized fluid withdrawal. When groundwater is withdrawn from a saturated soil, the effective stress on the soil skeleton is increased, resulting in consolidation and subsidence. Subsidence is known to have occurred in the Coachella Valley as a result of groundwater extraction (Sneed et al, 1998, 2000). Surveys conducted for the geothermal plants west of the site also suggest that groundwater extraction has caused several inches of subsidence in that area.

The site is also located within a region of active subsidence due to regional faulting. The Salton Trough is filled with up to 20,000 feet of Cenozoic-age sediments. Regional subsidence due to a combination of tectonic processes, including faulting and possible reservoir loading by the Salton Sea, may combine to produce roughly 1½ inches of settlement per year across the entire Salton Trough (Lofgren, 1978). Subsidence due to tectonic processes generally occurs over large areas.

8.0 CONCLUSIONS

It is our opinion that the proposed development is feasible from a geotechnical standpoint, provided that the recommendations in the following sections of this report are implemented. However, several geotechnical constraints exist which should be addressed prior to construction.

- Site improvements include heavy structures (such as the turbine generators and water tanks), as well as lightly loaded structures (such as the support building and minor equipment pads). For the lightly loaded structures, the potential for expansive soil heave will govern design considerations. For the heavy structures, the potential for settlement may govern design.

- Heavy structures such as the turbine generators and water tanks may either be founded on mat foundations or driven piles. In either case, the bottom of the mat foundation or the pile cap should be located at least 5 feet below finish grade. At this depth, we anticipate that the bearing soils will consist of hard fat clay with a dynamic shear modulus (G_{max}) of about 1,150 TSF, and a dynamic constrained modulus (E_s) of about 3,680 TSF. We have provided bearing capacities at this depth to limit the estimated total settlement to approximately 1 inch. If more bearing capacity is needed, deep foundations should be used. Alternative design parameters for square, precast, driven concrete piles are provided. The settlements associated with the allowable pile capacities should be within generally tolerable limits.

- Lightly loaded structures such as the support building and minor equipment pads may be founded on either conventional shallow foundations or post-tension slabs. If conventional shallow foundations are used, these structures should be underlain by at least 5 feet of select low expansion sand or gravel (expansion index less than 50). Alternatively, the upper 5 feet of on site clayey soil may be “moisture treated”, compacted, and used to support post-tension slab foundations. Post-tension slab foundation design parameters are provided assuming that the bearing soils will consist of highly expansive, moisture treated fat clay.

- Roughly two to four feet of loose alluvium and poorly consolidated lacustrine sediments exists at the site. These materials are considered to be susceptible to settlement from foundation or fill loads, or from hydro-compression associated with surface water infiltration. We recommend that the loose surficial soils be excavated and replaced as compacted fill prior to development throughout the proposed buildings and improvement areas.

- The surficial soils at the site include moderately to highly expansive clays (CL and CH). Heave of exterior slabs and sidewalks should be anticipated if these materials are used to support the planned improvements. To help reduce the potential for distress to the proposed flatwork, the upper two feet of exterior slab and sidewalk subgrade should be replaced with low expansion sand or gravel (expansion index less than 50).

- About one to two feet of sandy alluvium mantles the site. The alluvium is less expansive than the underlying fat clays associated with the lacustrine deposits. The sandy alluvium may be selectively excavated and stockpiled on site. The select sand may be used to construct the low expansion compacted fill mat recommended beneath the proposed exterior flatwork areas (2 feet deep), as well as the 5 foot deep compacted fill mat recommended beneath any conventional shallow foundations which may be constructed at the site.

- No groundwater was observed in the 18 explorations conducted on site, which were extended to depths greater than 50 feet below grade. However, it should be noted that perched groundwater may develop in the future due to changes in site drainage (such as the proposed retention basins), irrigation, or antecedent rainfall.

- One percolation test was conducted in each of the three proposed detention basins. The tests suggest that the lacustrine deposits have a low percolation rate (0 to ¼ gallons per square foot per day). The percolation test results are presented in Appendix C

- The potential for liquefaction at the site is currently low due to the lack of groundwater. As a minimum, a dynamic differential settlement of ¾ of on inch across the proposed structures should be accounted for in the structural design. If groundwater levels were to rise to within 20 feet of the ground surface within the life of the proposed structures, the potential may exist for liquefaction of the thinly bedded, discontinuous, saturated, granular soil layers. This could result in roughly 1 additional inch of total (½ inch differential) post-liquefaction settlement. Potential seismic hazards at the site should be mitigated through structural design in general accordance with the applicable codes.

9.0 RECOMMENDATIONS

The remainder of this report presents recommendations regarding earthwork construction and preliminary design of the proposed improvements. These recommendations are based on empirical and analytical methods typical of the standard of practice in southern California. If these recommendations do not to cover a specific feature of the project, contact our office for amendments.

9.1 Plan Review

We recommend that foundation and grading plans be reviewed by Geotechnics Incorporated prior to construction. It has been our experience that substantial changes in the development may occur from the preliminary plans used for the investigation. Such changes may require additional evaluation, which could result in modifications to the recommendations provided in the following sections of the report.

9.2 Excavation and Grading Observation

Foundation excavations and site grading excavations should be observed by Geotechnics Incorporated. During grading, Geotechnics Incorporated should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by the preliminary investigation, to adjust designs to actual field conditions, and to determine that the grading is accomplished in general accordance with the recommendations of this report. Recommendations presented in this report are contingent upon Geotechnics Incorporated performing such services. Our personnel should perform sufficient testing of fill during grading to support our professional opinion as to compliance with the compaction recommendations.

9.3 Earthwork

Grading and earthwork should be conducted in general accordance with the applicable grading ordinance and Appendix Chapter 33 of the Uniform Building Code. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on the conditions observed by our personnel during grading.

9.3.1 Site Preparation: Site preparation includes removal of deleterious materials, existing structures, or other improvements from areas to be subjected to fill or structural loads. Deleterious materials, including vegetation, trash, construction debris, and contaminated soils, should be removed from the site. Existing subsurface utilities that are to be abandoned should be removed and the excavations backfilled and compacted as described in Section 9.3.5.

9.3.2 Compressible Soils: The alluvium and surficial lacustrine sediments throughout the site are considered compressible should be excavated and compacted prior to construction of settlement sensitive improvements. Removals should expose competent lacustrine sediments as determined by our personnel during grading. In general, removals are anticipated to be on the order of 2 to 4 feet deep, although deeper removals may be required in some areas. Excavations bottoms should be observed by Geotechnics Incorporated prior to placement of compacted fill. The removed soil that is free of deleterious material may be replaced in accordance with Section 9.3.5 as a uniformly compacted fill to the proposed plan elevations.

9.3.3 Expansive Soils: Soil heave may cause distress to foundations, slabs, flatwork, and other improvements. Figure E-3 summarizes the expansion index testing conducted at the site. We anticipate that excavations will generate predominately clayey soils with a medium to high expansion potential. In order to reduce the anticipated soil heave, the upper two feet of soil (in exterior flatwork areas) and five feet of soil (for buildings on shallow foundations) should be excavated and replaced with low expansion soil or gravel (material with an expansion index less than 50). The remedial grading should include the area within five feet of the building perimeters. It should be noted that the surficial alluvium is anticipated to have a low expansion potential. The upper 1 to 2 feet of alluvium may be selectively excavated, stockpiled, and used to construct the select low expansion fill areas.

As an alternative to capping the building areas with 5 feet of low expansion soil, the expansive clay may be “moisture treated” to a depth of 5 feet, and post-tension slab foundations used for lightly loaded structures. Moisture treated fill should be brought to at least 5 percentage points above optimum moisture content, and then compacted to between 87 and 92 percent relative compaction, as described in Section 9.3.5.

9.3.4 Temporary Excavations: Temporary excavations are anticipated throughout the site for the removal of compressible materials and construction of the proposed utilities. Excavations should conform to Cal-OSHA guidelines. Temporary slopes should be inclined no steeper than 1:1 (horizontal to vertical) for heights up to 10 feet. Higher temporary slopes or excavations that encounter seepage should be evaluated by the geotechnical consultant on a case-by-case basis during grading.

9.3.5 Fill Compaction: All fill and backfill to be placed in association with site development should be accomplished at above optimum moisture conditions, and using equipment that is capable of producing a uniformly compacted product. The minimum relative compaction recommended for fill is 90 percent of the maximum dry density based on ASTM D1557, except as modified below.

If post-tension slabs are used for lightly loaded structures, we recommend that the on-site clays be “moisture treated” to at least 5 percentage points above optimum moisture, and then compacted to between 87 and 92 percent relative compaction based on ASTM D1557. Sufficient observation and testing should be performed by Geotechnics so that an opinion can be rendered as to the compaction achieved.

Imported fill sources should be observed prior to hauling onto the site to determine the suitability for use. In general, imported fill soils should have an expansion index less than 50 based on UBC Test Method 29-2 or ASTM D4829. Samples of imported materials should be tested by Geotechnics in order to evaluate their appropriate engineering properties for the planned use. During grading operations, soil types may be encountered by the contractor which do not appear to conform to those discussed within this geotechnical report. The geotechnical consultant should be contacted to evaluate the suitability of these soils for their proposed use.

9.3.6 Surface Drainage: Slope, foundation and slab performance depends greatly on how well surface runoff drains from the site. This is true both during construction and over the entire life of the structure. The ground surface around structures should be graded so that water flows rapidly away from the structures and slope tops without ponding. The surface gradient needed to achieve this may depend on the prevailing landscape. The project engineer should consider these aspects in design.

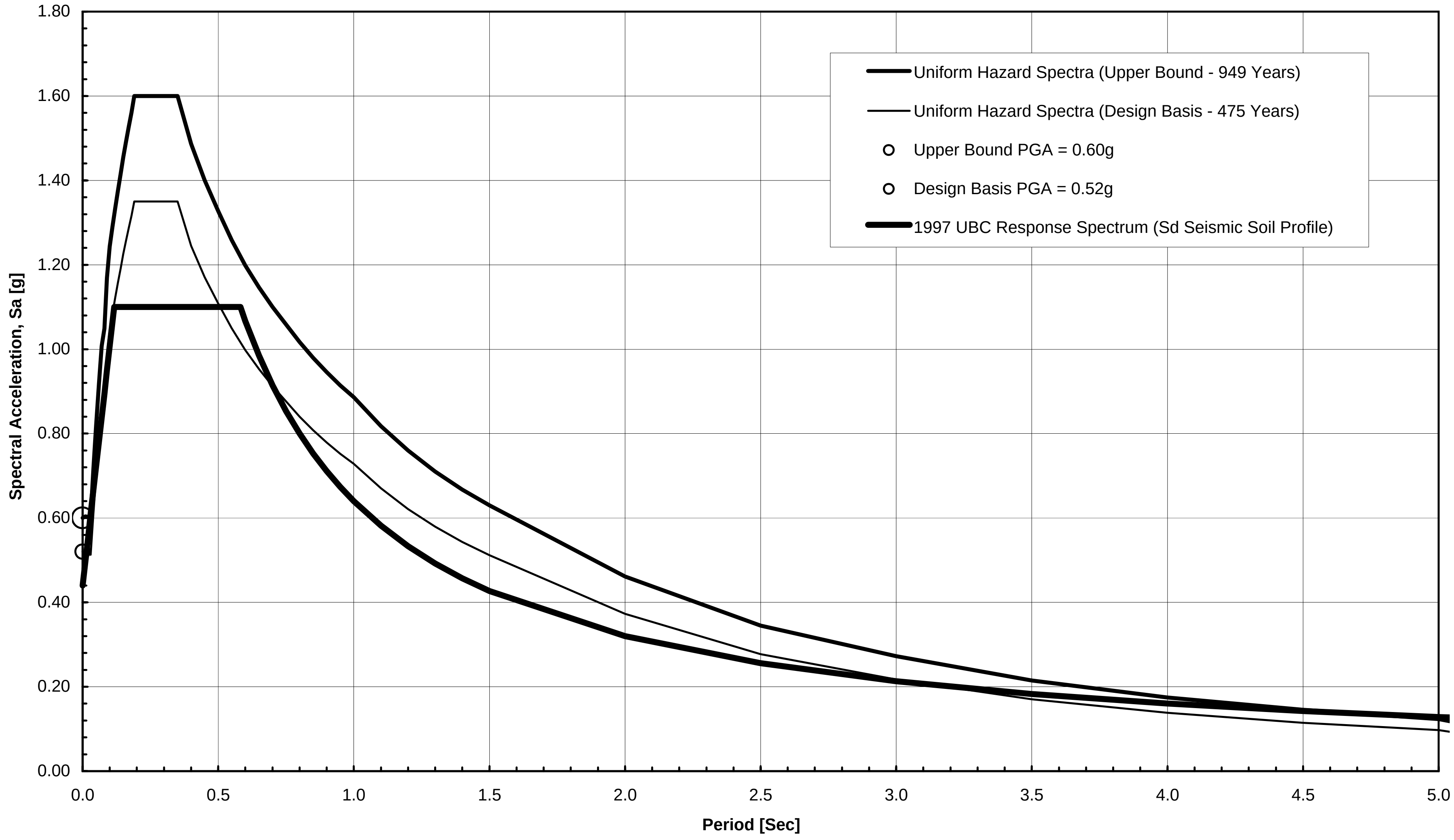
9.4 Shallow Foundations

Shallow foundations may be used for lightly loaded structures such as the proposed operations building and minor equipment pads. Shallow foundation design will be controlled by the potential for expansive soil heave. Our remedial grading recommendations for expansive soils were presented in Section 9.3.3. Conventional shallow foundations may be used for structures founded on at least 5 feet of low expansion sand or gravel. Post-tension slab foundations are recommended for structures founded on moisture treated fat clay.

The design of the foundation system should be performed by the project structural engineer, incorporating the following geotechnical parameters. These recommendations should be considered preliminary, and subject to revision based on the conditions observed during grading. They are only minimum criteria and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or the structural engineer.

9.4.1 Conventional Foundations: The following design parameters are appropriate for buildings underlain by at least 5 feet of compacted fill with a low expansion potential (an expansion index less than 50). The low expansion fill may consist of selectively graded on-site alluvium, or imported sand or gravel. The low expansion soil cap should extend at least five feet beyond the structural perimeter, and should be compacted to at least 90 percent relative compaction based on ASTM D1557.

Allowable Soil Bearing:	2,500 lbs/ft^2 (one-third increase for wind or seismic)
Minimum Footing Width:	12 inches
Minimum Footing Depth:	18 inches below lowest adjacent soil grade
Minimum Reinforcement:	Two No. 4 bars at both top and bottom
Subgrade Modulus:	150 lbs/in^3
Slab-on-Grade:	Slabs should be at least 6 inches thick, and reinforced with at least No. 3 bars on 18-inch centers, each way.



Geotechnics Incorporated

SPECTRAL ACCELERATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE 5

9.4.2 Post-Tension Slab Foundations: The following design parameters are appropriate for buildings underlain by 5 feet of moisture treated clay with an expansion index less than 120. The clay should be placed at a moisture content of five or more percentage points above optimum, and compacted to between 87 and 92 percent relative compaction. The following parameters were developed in general accordance with the design methodology of the Post-Tensioning Institute.

Edge Moisture Variation, e_m:	Center Lift:	6.0 feet
	Edge Lift:	3.0 feet
Differential Swell, y_m:	Center Lift:	4.5 inches
	Edge Lift:	1.0 inches
Allowable Bearing:	1,500 lb/ft^2 (at slab subgrade)	

9.4.3 Settlement: Total and differential settlements of the proposed shallow foundations from the recommended bearing capacities are not expected to exceed one inch, and three quarters of an inch, respectively. In addition to the static settlement estimates, foundations may experience dynamic differential settlements on the order of ¾ inch across the length of the structures, as described in Section 7.3.

9.4.4 Lateral Resistance: Lateral loads against the structure may be resisted by friction between the bottoms of footings and slabs and the supporting soil, as well as passive pressure from the portion of vertical foundation members embedded into compacted fill. A coefficient of friction of 0.35 and a passive pressure of 350 psf per foot of depth are recommended for conventional foundations on low expansion soil. A coefficient of friction of 0.25 and a passive pressure of 250 psf per foot of depth are recommended for post-tension slab foundations on moisture treated clay.

9.4.5 Seismic Design: Based on the shear wave velocity measurements conducted at the location of exploration BH-10, we recommend that a 1997 UBC Seismic Soil Profile S_D be used for general seismic design at the site (the average shear wave velocity (v_s) was 650 ft/s). The shear wave velocity measurements are summarized in Appendix B. The Brawley Seismic Zone, which is located 13 km from the site, is a Type B Seismic Source based on 1997 UBC criteria. The near source acceleration and velocity factors (N_a and N_v) both equal 1.0. The seismic coefficients C_a and C_v equal 0.44 and 0.64, respectively.

The 1997 UBC response spectrum for the site is presented in Figure 5. Site specific uniform hazard spectra corresponding to the *design basis* and *upper bound earthquakes* are also shown in Figure 5 for comparison. Structural design should comply with the requirements of the governing jurisdictions, building codes and standard practices of the Association of Structural Engineers of California.

9.5 Mat Foundations

Heavy structures such as the turbine generators and water tanks should either be founded on mat foundations or driven piles (see Section 9.6 for pile recommendations). The bottom of the mat foundation or the pile cap should be located at least 5 feet below finish grade. If mat foundations are less than 5 feet thick, the material between the bottom of the mat and a depth of 5 feet should consist of aggregate base compacted to at least 95 percent relative compaction based on ASTM D1557. Note that no additional remedial grading is anticipated for mat foundations (the potentially compressible and actively expansive soils should be removed by the mat foundation excavations). At depths of 5 feet or more, we anticipate that the bearing soils will consist of hard fat clay with a dynamic shear modulus (G_{max}) of about 1,150 TSF, and a dynamic constrained modulus (E_s) of about 3,680 TSF. Mat foundation design may be based on the following design parameters.

Allowable Soil Bearing:	4,000 lbs/ft^2 (one-third increase for wind or seismic)
Minimum Dimensions:	5 feet wide by 5 feet long
Minimum Depth:	5 feet below lowest adjacent soil grade
Subgrade Modulus:	100 lbs/in^3
Differential Settlement:	¾ inch

9.5.1 Settlement: Total and differential settlements of the proposed mat foundations from the recommended bearing capacity are not expected to exceed one inch, and three quarters of an inch, respectively. If more bearing is needed, the settlement will increase, and deep foundations should be considered. Pile recommendations are presented in Section 9.6. In addition to the static settlement estimates described above, mat foundations may experience dynamic differential settlements on the order of ¾ inch across the length of the mat, as described in Section 7.3.

9.6 Deep Foundations

As an alternative to mat foundations, deep foundations may be used to support the proposed turbine generators and water tanks. We have conducted pile analyses using the CPT data, and assuming that driven, precast, square concrete piles will be used. For our analyses, pile diameters of 12, 14 and 16-inches were assumed. The bottom of the pile cap was assumed to be located 5 feet below the ground surface. Piles were assumed to be spaced at 3 feet in each direction (group effects were neglected). The estimated pile capacities at each CPT sounding location are presented in Appendix G.

9.6.1 Axial Capacity: The gross axial capacity (Q_{ug}) of each individual pile will be the sum of the pile tip resistance (Q_p) and the skin friction (Q_s) accumulated along the length of the pile (skin friction dominates). Estimated gross axial pile capacities for 20 and 80 foot deep piles are presented below. The assumed minimum pile spacing (three pile diameters) should result in negligible group effects for axial loads. The net axial capacity (Q_{un}) of each pile will equal the gross axial capacity minus the tributary weight of the piles and pile cap (W_{p+c}).

$$Q_{un} = Q_{ug} - W_{p+c} = (Q_p + Q_s) - W_{p+c}$$

PILE DEPTH	12-INCH CAPACITY	14-INCH CAPACITY	16-INCH CAPACITY
20 Feet	60 Kips	75 Kips	90 Kips
80 Feet	144 Kips	171 Kips	198 Kips

The allowable gross axial pile capacities presented in the table above are consistent with the equations shown for the CPT sounding at BH-10 (see Figure G-3 in Appendix G). The allowable gross axial capacities of the various piles are:

$$Q_{ug} \sim 1.4 * (Z - 20) + 60 \text{ Kips (for a 12-inch pile)}$$
$$Q_{ug} \sim 1.6 * (Z - 20) + 75 \text{ Kips (for a 14-inch pile)}$$
$$Q_{ug} \sim 1.8 * (Z - 20) + 90 \text{ Kips (for a 16-inch pile)}$$

Note that the allowable gross axial pile capacities incorporate a safety factor of approximately 2. A one-third increase in the pile capacity may be used when considering short-term wind and seismic loads. The compressive strength of the pile section should be verified by the project structural engineer.

It should be noted that pile foundations do not mitigate the dynamic settlement hazard. We estimate that a total dynamic settlement of ½ to 1½ inches may occur at the site. Current design philosophies (the neutral-plane approach) suggest that such settlement will not decrease the axial pile capacity. Instead, the pile may experience increased internal stress and undergo a small fraction of the total dynamic settlement. The pile capacities presented above were not reduced to reflect dragload.

We recommend that at least one test pile be installed at the site to confirm the pile capacity estimates presented above. Test piles should be driven under the observation of Geotechnics Incorporated. The pile driving criteria and final axial capacity should be based on driving conditions encountered and the pile hammer used. If difficult driving is encountered, pre-drilling may be used. The area of the pre-drilled hole should not exceed 80 percent of the cross-sectional area of the pile. Piles should not be installed until the required concrete compressive strength has been achieved, as determined by the structural engineer.

9.6.2 <u>Uplift Capacity:</u> The net uplift capacity (T_{un}) of each individual pile will be controlled by the skin friction. The gross uplift capacity (T_{ug}) will equal the individual uplift capacity plus the weight of the pile and pile cap (W_{p+c}). Estimated net uplift capacities for 20 and 80 foot deep piles are shown below. Depending upon the ultimate pile configuration, a group reduction factor (η_T) may apply.

$$T_{ug} = T_{un} * \eta_T + W_{p+c}$$

PILE DEPTH	12-INCH CAPACITY	14-INCH CAPACITY	16-INCH CAPACITY
20 Feet	24 Kips	27 Kips	30 Kips
80 Feet	108 Kips	123 Kips	138 Kips

Linear approximations of the allowable net uplift capacities are presented below. These values incorporate a safety factor of approximately 2 or more. The tensile strength of the pile section should be verified by a structural engineer.

$T_{un} \sim 1.4 * (Z - 20) + 24$ Kips (for a 12-inch pile)
$T_{un} \sim 1.6 * (Z - 20) + 27$ Kips (for a 14-inch pile)
$T_{un} \sim 1.8 * (Z - 20) + 30$ Kips (for a 16-inch pile)

9.6.3 Lateral Pile Capacity: The program LPILE $^{\text{Plus}}$ 4.0 was used to conduct lateral pile analyses for single piles. The piles were assumed to be loaded to the estimated axial capacity (Q_{ug}) presented in Section 9.6.1 (the maximum axial loads govern deflection). The pile caps were assumed to consist of a fixed head condition (zero rotation). The lateral load at the cap was varied until the displacement equaled approximately ¼ to ½ inch. The corresponding axial loads are presented below.

PILE CAP DISPLACEMENT	12-INCH PILE LOAD	14-INCH PILE LOAD	16-INCH PILE LOAD
¼ Inch	21 Kips	26 Kips	32 Kips
½ Inch	30 Kips	38 Kips	46 Kips

In addition to the lateral load capacity of the piles, lateral loads may be resisted by friction between the bottom of pile cap and the supporting soil, as well as passive pressure from the embedded portion of pile cap. A coefficient of friction of 0.25 and a passive pressure of 250 psf per foot of depth are recommended. The lateral capacity developed by friction and passive pressure may be added to that presented in the table above for approximately the same total pile cap displacement.

9.6.4 Settlement: The program TZPILE was used to estimate pile settlement at the site. We estimate that piles loaded to the allowable axial capacities presented in Section 9.6.1 will experience less than ¼ inch total settlement. In addition, dynamic settlements on the order of ½ to 1½ inches may occur around the structure, as described in Section 7.3. A small fraction of the dynamic settlement may be transmitted to the piles. The remaining dynamic settlement will manifest as differential movement between the pile cap and surrounding soil.

9.7 On-Grade Slabs

The project structural engineer should design the proposed slabs for the anticipated loading using the following minimum geotechnical parameters. On-grade slabs should be supported by compacted fill prepared as recommended in Section 9.3. If an elastic design is used, a modulus of subgrade reaction of 100 lb/in^3 would be appropriate. Building slabs should be at least 6 inches thick with at least No. 3 bars on 18 inch centers, each way. Reinforcement should be placed near the top of the slab with at least 1½ inches cover.

9.7.1 Moisture Protection for Slabs: Concrete slabs constructed on grade ultimately cause the moisture content to rise in the underlying soil. This results from continued capillary rise and the termination of normal evapotranspiration. Because normal concrete is permeable, the moisture will eventually penetrate the slab. Excessive moisture may cause mildewed carpets, lifting or discoloration of floor tiles, or similar problems. To decrease the likelihood of problems related to damp slabs, suitable moisture protection measures should be used where moisture sensitive floor coverings, moisture sensitive equipment, or other factors warrant.

The most commonly used moisture barriers in southern California consist of two to four inches of clean sand or pea gravel covered by 'visqueen' plastic sheeting. Two inches of sand are placed over the plastic to decrease concrete curing problems. It has been our experience that such systems will transmit approximately 6 to 12 pounds of moisture per 1000 square feet per day. The architect should review the estimated moisture transmission rates, since these values may be excessive for some applications, such as sheet vinyl, wood flooring, vinyl tiles, or carpeting with impermeable backings that use water soluble adhesives. Sheet vinyl may develop discoloration or adhesive degradation due to excessive moisture. Wood flooring may swell and dome if exposed to excessive moisture. The architect should specify an appropriate moisture barrier based on the allowable moisture transmission rate for the flooring. This may require a “vapor barrier” rather than a “vapor retarder”.

The American Concrete Institute provides detailed recommendations for moisture protection systems (ACI 302.1R-04). ACI defines a “vapor retarder” as having a minimum thickness of 10-mil and a water transmission rate of less than 0.3 perms when tested in accordance with ASTM E96. ACI defines a “vapor barrier” as having a water transmission rate of 0.0 perms. The vapor membrane should be constructed in accordance with ASTM E1643 and E1745 guidelines. All laps or seams should be overlapped a minimum of 6 inches, or as recommended by the manufacturer. Joints and penetrations should be sealed with pressure sensitive tape, or the manufacturer’s recommended adhesive. The vapor membrane should be protected from puncture, and repaired per the manufacturer’s recommendations if damaged. The project architect should review ACI 302.1R-04 along with the moisture requirements of the proposed flooring system, and incorporate an appropriate level of moisture protection as part of the flooring design.

The vapor membrane is often placed over 4 inches of a granular base material. The base should be a clean, fine graded sandy material with at 10 to 30 percent passing the No. 100 sieve. The base should not be contaminated with clay, silt, or organic material. The base should be proof-rolled prior to placing the vapor membrane.

Based on current ACI recommendations, concrete should be placed directly over the vapor membrane. The common practice of placing sand over the vapor membrane may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor membrane may also move and mound prior to concrete placement, resulting in an irregular slab thickness. When placing concrete directly on an impervious membrane, it should be noted that finishing delays may occur. Care should be taken to assure that a low water to cement ratio is used, that the concrete is moist cured in accordance with ACI guidelines.

9.7.2 Exterior Slabs: Because of the presence of expansive soils throughout the site, differential heave of exterior flatwork should be anticipated. One inch of differential heave is not considered unusual, and more may occur. The potential for heave and distress may be reduced by excavating the upper two feet of clayey subgrade, and replacing with a low expansive sand (EI<50). As a minimum, we recommend that the upper two feet of subgrade materials be brought to at least 5 percentage points above optimum moisture content immediately prior to placement of concrete.

Exterior slabs should be at least 4 inches thick. Crack control joints should be placed on a maximum spacing of 10 foot centers, each way, for slabs, and on 5 foot centers for sidewalks. The potential for long-term differential movements across the control joints may be reduced by using steel reinforcement. Exterior slabs constructed on expansive clay should be reinforced with at least 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the slab section.

9.7.3 Expansive Soils: The surficial soils observed during our investigation primarily consisted of sandy clays (CL) and fat clays (CH) with a medium to high expansion potential. The expansion index test results are presented in Figure E-3. Mitigation alternatives for expansive soils were discussed in Section 9.3.3.

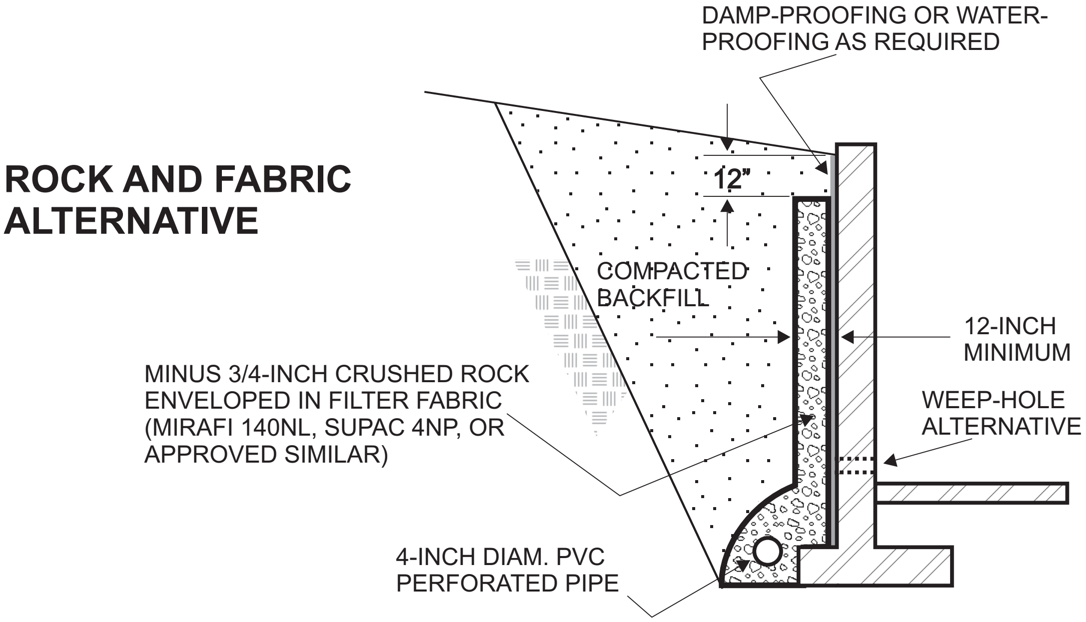
9.7.4 Reactive Soils: In order to assess the exposure of concrete in contact with the site soils, samples were tested for water soluble sulfate content (see Figure E-4). The tests indicate that the site soils present a *severe sulfate exposure* based on UBC criteria. According to Table 19-A-4 of the 1997 UBC, all concrete which will come in contact with the pore fluid generated from the site soils (including foundations and slabs) should be designed to reduce the potential for long term sulfate degradation. UBC Table 19-A-4 indicates that Type V cement should be used with a maximum water to cement ratio of 0.45, and a 28-day compressive strength of 4,500 psi.

In order to assess the reactivity of the site soils with metal pipe, the pH and resistivity of selected soil samples was determined. The test results are also summarized in Figure E-4. The test results suggest that the site soils are corrosive to metal pipes. A corrosion engineer should be contacted for specific recommendations. Additional field resistivity testing was conducted by Schiff Associates (see Appendix D).

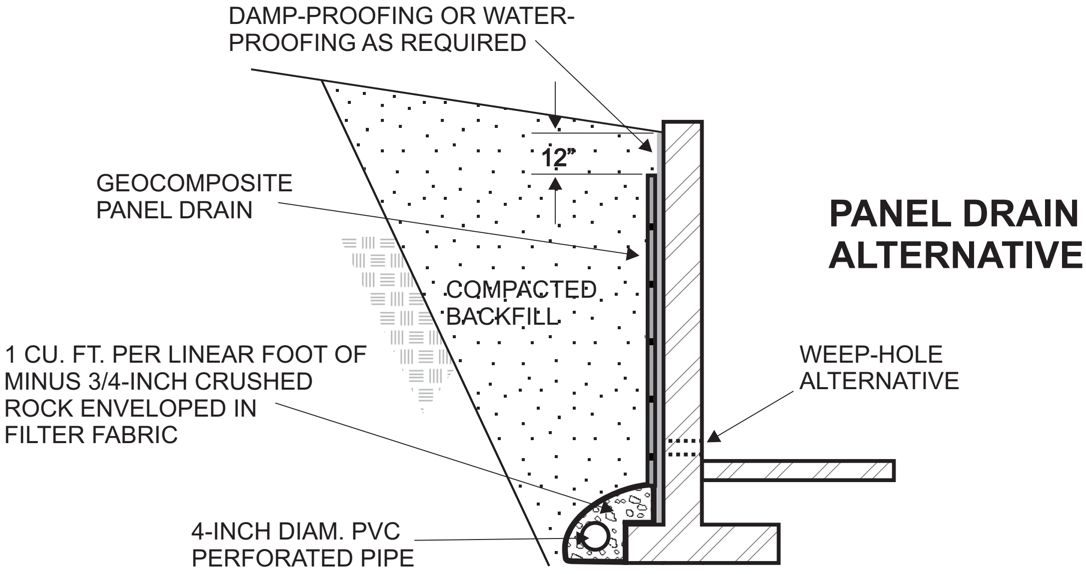
9.8 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active or at-rest pressures. We recommend that retaining walls be backfilled with soil which has an expansion index of 20 or less. *The on site soils do not meet this criterion*. Retaining wall backfill should be compacted to at least 90 percent relative compaction, based on ASTM D1557. Backfill should not be placed until walls have achieved adequate structural strength. Heavy compaction equipment, which could cause distress to the walls, should not be used. Walls should contain backdrains to relieve hydrostatic pressure. Our recommended wall drain details are shown in Figure 6.

For general wall design, an allowable bearing capacity of 2,000 lbs/ft^2, a coefficient of friction of 0.25, and a passive pressure of 250 psf per foot of depth is recommended. Wall footings should be embedded at least 24 inches below lowest adjacent soil grade. Cantilever retaining walls with level granular backfill may be designed using an active earth pressure approximated by an equivalent fluid pressure of 35 lbs/ft^3. These active pressures should be used for walls free to yield at the top at least one percent of the wall height. Walls that are restrained so that such movement is not permitted, or walls with 2:1 sloping backfill should be designed for an active earth pressure approximated by an equivalent fluid pressure of 55 lbs/ft^3. Note that these pressures do not include the effects of surcharge loads.



ROCK AND FABRIC ALTERNATIVE



PANEL DRAIN ALTERNATIVE

NOTES

1) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.
2) As an alternative to the perforated pipe and outlet, weep-holes may be constructed. Weep-holes should be at least 2 inches in diameter, spaced no greater than 8 feet, and be located just above grade at the bottom of wall.
3) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599, or similar approved fabric. Filter fabric should be overlapped at least 6-inches.
4) Geocomposite panel drain should consist of Miradrain 6000, J-DRain 400, Supac DS-15, or approved similar product.
5) Drain installation should be observed by the geotechnical consultant prior to backfilling.

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WALL DRAIN DETAILS

Project No. 0554-075-00
Document No. 06-0015

FIGURE 6

9.9 Pavement Design

Alternatives for either asphalt concrete or Portland cement concrete pavements are given below. In both cases, we recommend that the upper 12 inches of pavement subgrade be scarified immediately prior to constructing the pavement section, brought to above optimum moisture content and compacted to at least 90 percent of the maximum dry density (ASTM D1557). Aggregate base should be compacted to at least 95 percent relative compaction, and should conform to Section 26 of the Caltrans Standard Specifications or Section 200-2 of the *Standard Specifications for Public Works Construction (SSPWC).* Asphalt concrete should conform to Section 26 of the Caltrans Standard Specifications or Section 400-4 of the *SSPWC*. Asphalt concrete should be compacted to at least 95 percent relative compaction based on the Hveem density.

9.9.1 Asphalt Concrete: The following preliminary pavement sections are provided for estimation purposes only. Three traffic indices were assumed for preliminary design (TI of 5.0, 6.0 and 7.5). The project civil engineer should review the assumed traffic indices to determine if and where they are appropriate for use at the site.

R-Value testing was conducted on two samples taken during our investigation in general accordance with CTM 301. During grading, samples of the actual pavement subgrade may be tested for R-Value, and the pavement sections refined throughout the site. Asphalt concrete pavement design was conducted in general accordance with the Caltrans Design Method (Topic 608.4). Based on the assumed traffic indices, and using the minimum R-Value of 5 determined in the laboratory, the following preliminary pavement sections are recommended.

TRAFFIC INDEX	**ASPHALT SECTION**	**BASE SECTION**
7.5	4 Inches	18 Inches
6.0	4 Inches	12 Inches
5.0	3 Inches	10 Inches

9.9.2 Portland Cement Concrete: Concrete pavement design was conducted in accordance with the simplified design procedure of the Portland Cement Association. This methodology is based on a 20 year design life. We assumed interlock would be used for load transfer across control joints. The subgrade materials were assumed to provide "low" subgrade support based on the results of the R-Value testing. Furthermore, the portland cement concrete was assumed to have a minimum 28 day flexural strength of 600 psi. Based on these assumptions, and using the same traffic indices presented previously, we recommend that the PCC pavement sections at the site consist of at least 6½ inches of concrete placed directly over compacted soil. For heavy traffic areas, we recommend using 7 inches of concrete placed over 6 inches of aggregate base. Crack control joints should be constructed for all PCC pavements on a maximum spacing of 10 feet, each way. Concentrated truck traffic areas should be reinforced with number 4 bars on 18-inch centers, each way.

9.10 Pipelines

It is our understanding that the proposed development will include a variety of pipelines such as storm drains and sewers. Geotechnical aspects of pipeline design include lateral earth pressures for thrust blocks, modulus of soil reaction, and pipe bedding. Each of these parameters is discussed separately below.

9.10.1 Thrust Blocks: Lateral resistance for thrust blocks may be determined by a passive pressure value of 250 lbs/ft^2 for every foot of embedment, assuming a triangular pressure distribution. This value may be used for thrust blocks embedded into compacted fill or formation.

9.10.2 Pipe Bedding: Typical pipe bedding as specified in the *Standard Specifications for Public Works Construction* may be used. As a minimum, we recommend that pipes be supported on at least 4 inches of granular bedding material such as minus 3/4-inch crushed rock or disintegrated granite.

9.10.3 Modulus of Soil Reaction: The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines. For the purpose of evaluating deflection due to the load associated with trench backfill over the pipe, a value of 1,500 lbs/in^2 is recommended for the general site conditions, assuming granular bedding material is placed around the pipe.

10.0 LIMITATIONS OF INVESTIGATION

This investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No warranty, express or implied, is made as to the conclusions and professional opinions included in this report. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated into the plans, and the necessary steps are taken to see that the contractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

***

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APPENDIX A

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APPENDIX B

SUBSURFACE EXPLORATION

Field exploration consisted of a visual and geologic reconnaissance of the site, the drilling of 12 exploratory borings, and the advancement of 6 cone penetrometer (CPT) soundings. The maximum depth of exploration was approximately 91 feet. The approximate locations of the borings and CPT soundings are shown on the Exploration Plan, Figure 2. Logs describing the subsurface conditions encountered are presented in the following Figures B-1 through B-18.

The 12 exploratory borings were drilled to a maximum depth of 51½ feet between January 12 and 17 using a truck mounted, 8-inch diameter, continuous flight, hollow stem, auger drill rig. Disturbed soil samples were collected from the borings using a Standard Penetration Test (SPT) sampler (2-inch outside diameter). Relatively undisturbed samples were collected using a 3-inch outside diameter, ring lined sampler (modified CALifornia sampler). The SPT and CAL samples were sealed in plastic bags, labeled, and returned to the laboratory for testing. The drive weight for these samples was a 140-pound hammer with a free fall of 30 inches. For each sample, the number of blows needed to drive the sampler 12 inches was recorded on the logs under "blows per ft." Standard Pen samples are indicated on the boring logs with "SPT", and modified California samples with "CAL". Bulk soil samples are indicated on the logs with shading.

The 6 cone penetrometer (CPT) soundings were advanced to a maximum depth of 91 feet by Kehoe Testing and Engineering on January 16 and 17, 2006. The CPT soundings were advanced using a 30-ton truck mounted rig with a 15 cm^2 cone. The soundings were conducted in general accordance with ASTM method D5778. Integrated electronic circuitry was used to measure the tip resistance (Qc) and skin friction (Fs) at 2.5 cm (1 inch) intervals while the CPT was advanced into the soil with hydraulic down pressure. The data from the CPT soundings is presented in Figures B-2, B-7, B-10, B-11, B-14 and B-15. For each CPT sounding, the soil interpretation as a function of the normalized cone resistance and friction ratio is presented (Robertson, 1990). The soil interpretations are also shown in a color coded log on the final figure for each CPT sounding.

For one of the CPT soundings, shear wave velocity measurements were made at about 5 foot intervals. The shear waves were generated using an air actuated hammer located inside the front jack of the CPT rig. The shear wave arrival times were measured using a triaxial geophone located near the cone tip. The shear wave velocity measurements are discussed in the text of this report.

APPENDIX B

SUBSURFACE EXPLORATION (Continued)

The exploration locations were provided by the Imperial Irrigation District, as shown on the *Boring Location Plan, Drawing No. C1-2.* The latitude and longitude of the borings and CPT soundings were located in the field using a hand held GPS receiver. The locations shown should not be considered more accurate than is implied by the method of measurement used and the scale of the map. The lines designating the interface between differing soil materials on the logs may be abrupt or gradational. Further, soil conditions at locations between the excavations may be substantially different from those at the specific locations explored. It should be noted that the passage of time can result in changes in the soil conditions reported in our logs.

LOG OF EXPLORATION BORING NO. BH-1

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/13/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
			▨			**ALLUVIUM:** Well graded sand (SW), moderate yellowish brown, fine to coarse, very loose, trace of gravel.	Gradation Hydrometer
1			▨			Poorly graded sand (SP), moderate yellowish brown, fine ,dry to moist, very loose.	Atterberg Limits Soluble Sulfate
2			▨			**LACUSTRINE DEPOSITS:** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, trace of silt (ML), iron oxide staining, salt deposits.	Soluble Chloride pH & Resistivity Expansion Index
3							
4							
5	9	SPT					Gradation
6							
7							
8							
9							
10	35	CAL		105	21		
11							
12							
13							
14							
15	18	SPT				Trace of sandy silt (ML), pale yellowish brown, fine, dry to moist.	Gradation
16							
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-1(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/13/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	37	CAL		107	22	**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, trace of sandy silt.	Consolidation
22							
23							
24							
25							
26	14	SPT				Pocket penetrometer (PP) > 4.5 tons per square foot (tsf).	Gradation
27							
28							
29							
30							
31	30	CAL		107	21	PP > 4.5 tsf.	Consolidation
32							
33							
34							
35							
36	19	SPT				Interbedded layers of sandy silt (ML), pale yellowish brown, fine, dry to moist, between 5/8" to 2" thick. PP > 4.5 tsf.	Gradation
37							
38							
39							
40							

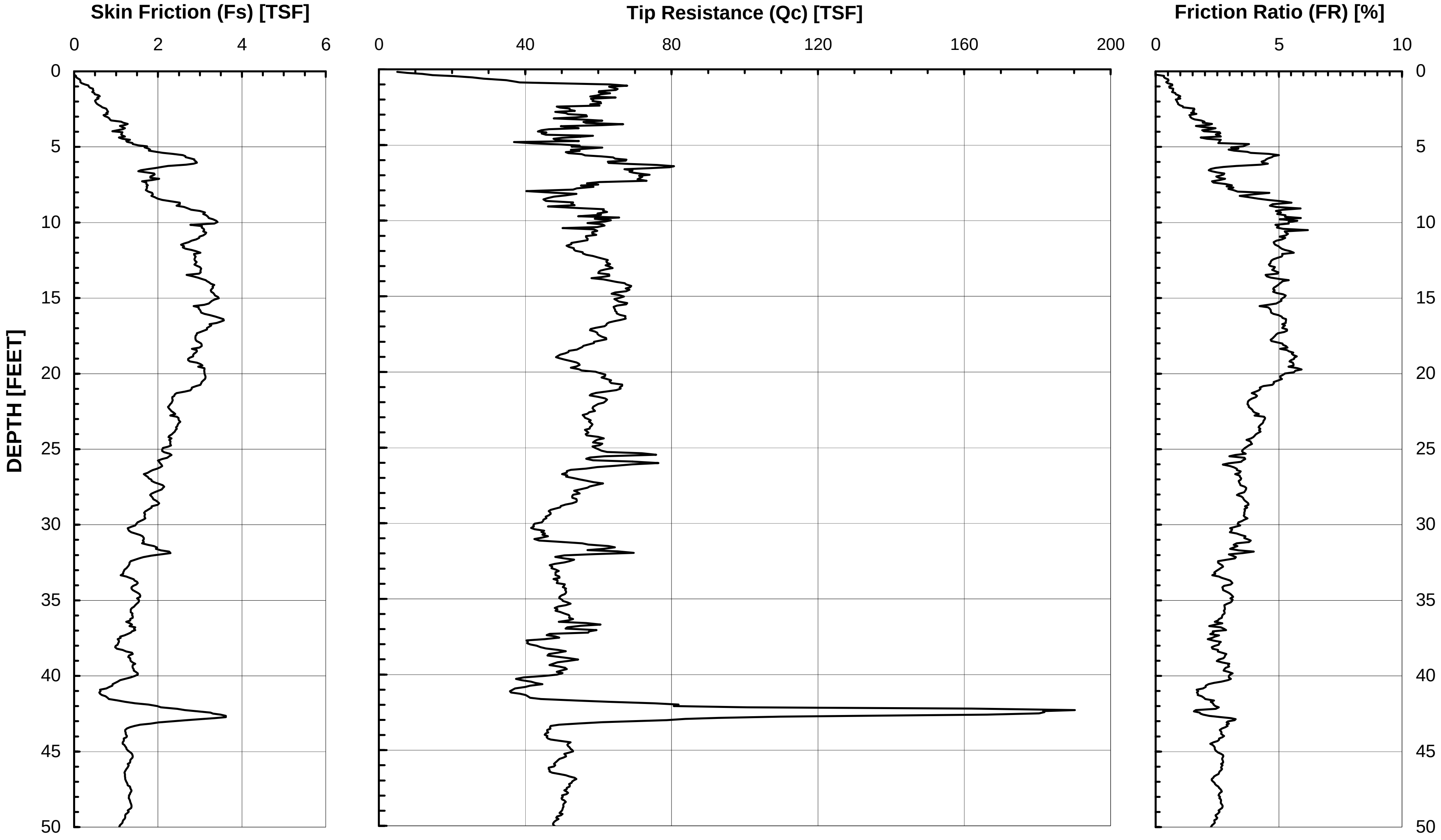
LOG OF EXPLORATION BORING NO. BH-1(continued)

Logged by: **JSO**

Method of Drilling: **8-inch diameter hollow-stem auger** **Date Drilled:** **1/13/2006**

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
41	18	SPT				**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, trace of sandy silt. PP > 4.5 tsf.	
42							
43							
44							
45						Greenish gray. PP = 2.5 tsf.	Gradation
46	13	SPT					
47							
48							
49							
50						Interbedded layers of fat clay (CH) dark yellowish brown, high plasticity, moist, and silty lean clay (CL), greenish gray, medium plasticity, moist, hard. PP = 2.5 tsf.	
51	18	SPT					
52						Total depth: 51½ feet No groundwater encountered	
53							
54							
55							
56							
57							
58							
59							
60							

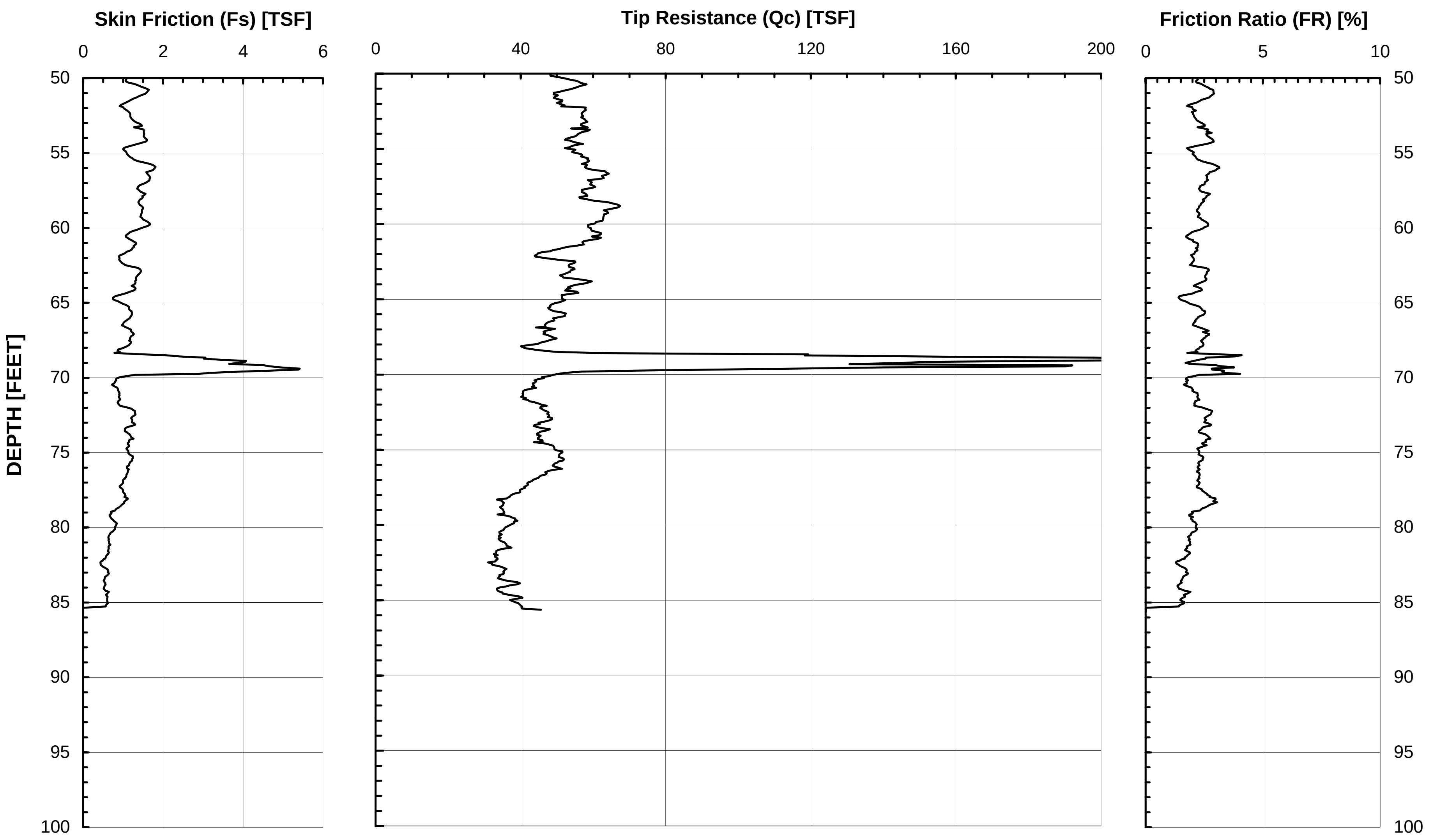
PROJECT NO. 0554-075-00 **GEOTECHNICS INCORPORATED** **FIGURE B-1.3**



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-2)

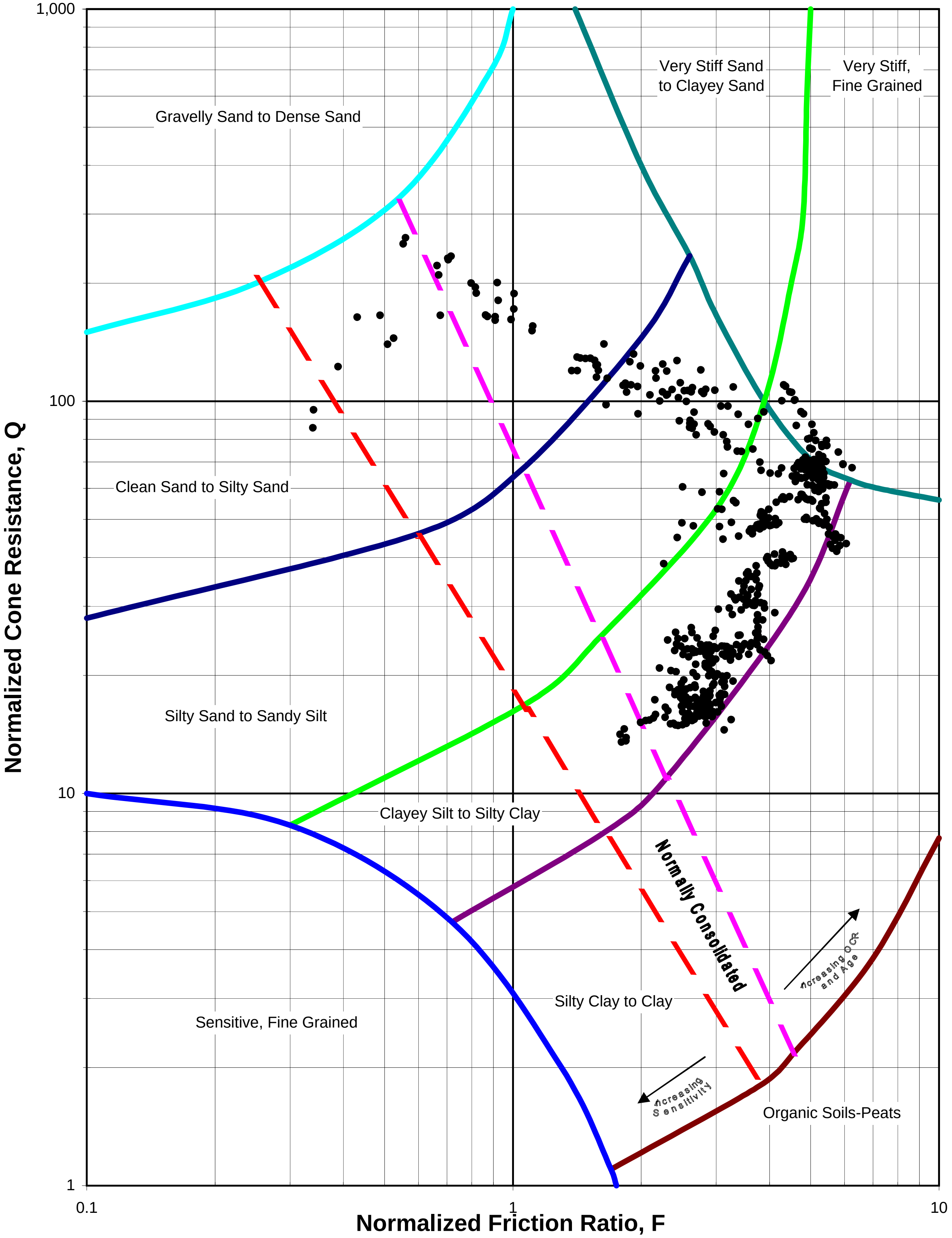
Project No. 0554-075-00
Document No. 06-0015
FIGURE B-2.1



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-2)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-2.2

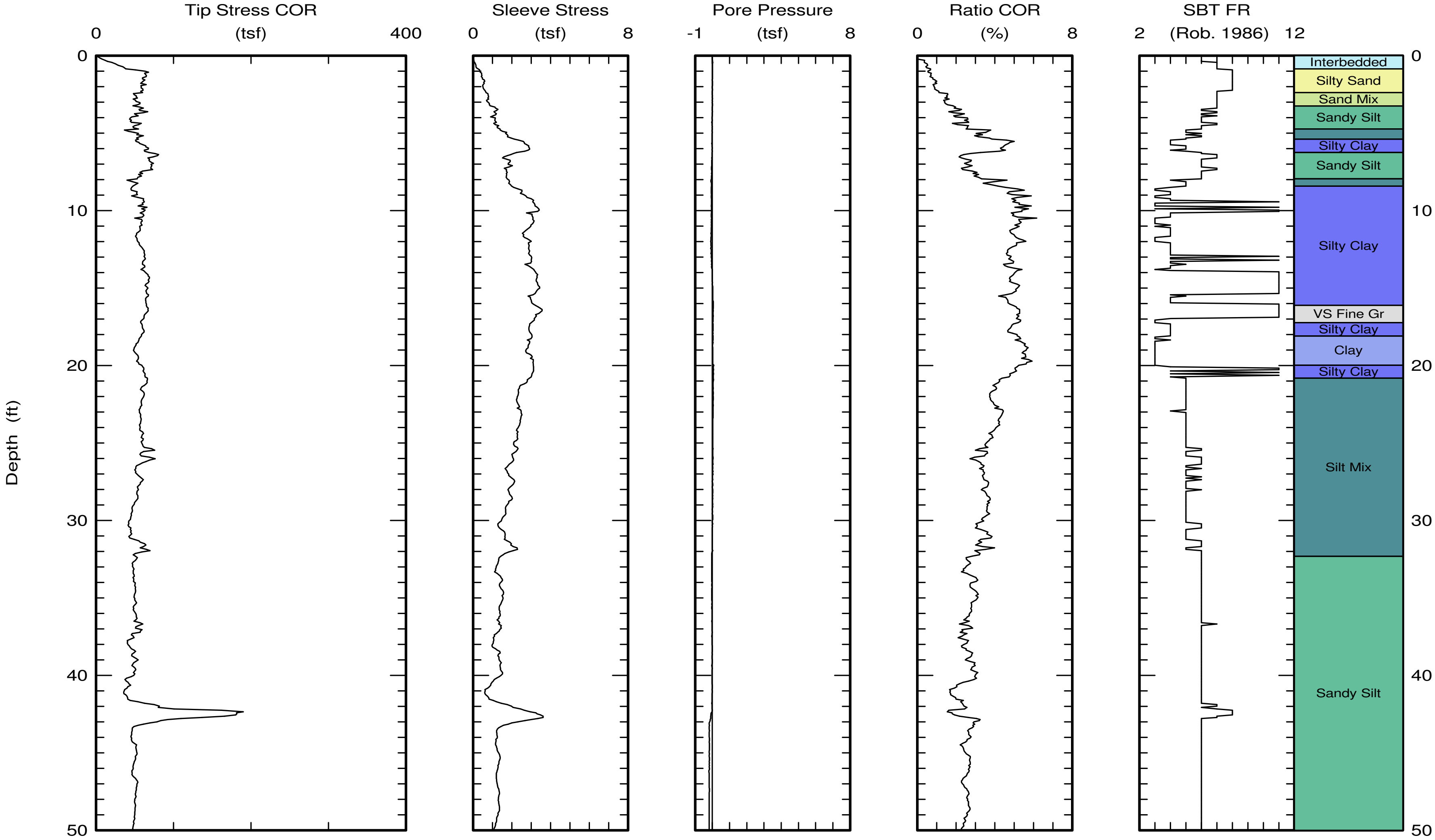


Geotechnics Incorporated

SOIL CLASSIFICATION (CPT-2)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-2.3

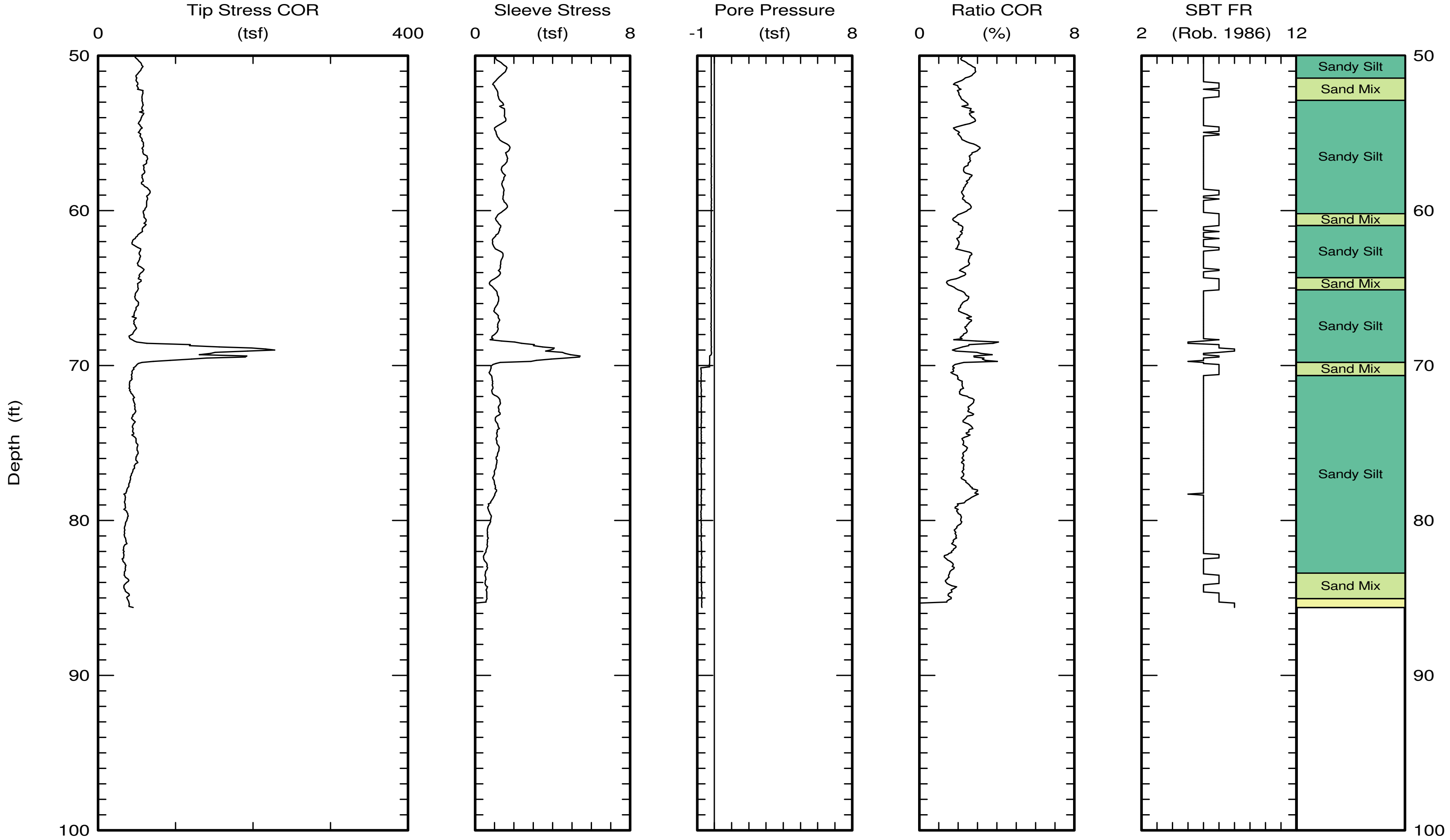
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 16/Jan/2006 Test ID: BH-2 Project: Niland
	Client: Geotechnics Inc Job Site: Niland Power Substation	



Maximum depth: 85.62 (ft)

Page 1 of 2

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 16/Jan/2006 Test ID: BH-2 Project: Niland
	Client: Geotechnics Inc Job Site: Niland Power Substation	



Maximum depth: 85.62 (ft)

Page 2 of 2

LOG OF EXPLORATION BORING NO. BH-3

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/13/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
			▨			**ALLUVIUM:** Poorly graded sand with silt and gravel (SP-SM), moderate yellowish brown, fine to coarse, dry, very loose.	Gradation Hydrometer Atterberg Limits Maximum Density Optimum Moisture Expansion Index R-Value
1			▨				
2			▨			**LACUSTRINE DEPOSITS:** Fat clay with sand (CH), moderate yellowish brown, dry to moist, high plasticity, trace of gravel.	
3			▨				
4			▨				
5			▨				
6	35	CAL		106	18	Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, salt deposits.	
7							
8							
9							
10							
11	10	SPT				Thinly laminated beds of sandy silt (ML), pale yellowish brown to dark yellowish orange, fine, dry to moist, approximately 5/8" thick. PP > 5.0 tsf.	
12							
13							
14							
15						Lens of sandy silt (ML), pale yellowish brown and dark yellowish orange.	
16	41	CAL		106	20		
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-3(continued)

Logged by: **JSO**

Method of Drilling: **8-inch diameter hollow-stem auger** **Date Drilled:** **1/13/2006**

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	14	SPT				**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, lens of sandy silt.	
22							
23							
24							
25							
26	36	CAL		106	21	Interbedded layers of sandy silt (ML), pale yellowish brown, fine, dry to moist, between ½" to 1" thick.	
27							
28							
29							
30							
31	19	SPT				PP > 4.5 tsf.	
32						Total depth: 31½ feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							

LOG OF EXPLORATION BORING NO. BH-4

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/12/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**LACUSTRINE DEPOSITS:** Sandy lean clay (CL), moderate yellowish brown, medium plasticity.	
2							
3							
4							
5							
6	16	SPT				Fat clay (CH), dark yellowish brown, high plasticity, dry to moist, hard, with interbedded fat clay and sandy silt (CH and ML), pale yellowish brown and dark yellowish orange, fine, dry to moist, iron oxide staining, between ½" to ¾" thick.	
7							
8							
9							
10							
11	48	CAL		107	15		
12							
13							
14							
15							
16	16	SPT				Some thin silt interbeds.	
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-4(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/13/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	66	CAL		109	18	**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, interbedded fat clay and sandy silt (CH and ML), pale yellowish brown and dark yellowish orange between ½" to ¾" thick.	
22							
23							
24							
25							
26	14	SPT					
27							
28							
29							
30							
31	40	CAL		105	18		
32						Total depth: 31 feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							

LOG OF EXPLORATION BORING NO. BH-5

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/12/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**LACUSTRINE DEPOSITS:** Sandy clay (CL), moderate yellowish brown, dry to moist, medium plasticity, trace gravel.	
2			▨				
3			▨				
4			▨				
5			▨				
6	17	CAL		103	9		
7							
8							
9						Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, interbedded layers of sandy silt (ML), greenish gray and dark yellowish orange, fine, dry to moist, approximate 2" lens of greenish gray silt at 11½ feet.	
10							
11	16	SPT					
12							
13							
14							
15							
16	73	CAL		114	15		
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-5(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/12/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	17	SPT				**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, interbedded fat clay and sandy silt (CH and ML), approximately 1" thick.	
22							
23							
24							
25							
26	16	SPT					
27							
28							
29							
30							
31	15	SPT				Interbedded sandy silt (ML) and silty sand (SM), yellowish brown, fine, moist, medium dense, approximately ½" thick.	
32							
33							
34							
35							
36	15	SPT					
37							
38							
39							
40						-- Sandy silt (ML), medium to light gray, fine, moist, medium dense.	

LOG OF EXPLORATION BORING NO. BH-5(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/13/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
41	20	SPT				**LACUSTRINE DEPOSITS: (continued)** Sandy silt (ML), medium light gray, fine, moist, medium dense.	
42							
43							
44							
45						Fat clay (CH), dark yellowish brown, high plasticity, moist, hard.	
46	13	SPT				PP = 2.5 tsf.	
47							
48							
49							
50						Interbedded sandy silt beds (ML), medium light gray, less than 1" thick.	
51	19	SPT				PP > 5.0 tsf.	
52						Total depth: 51½ feet	
53						No groundwater encountered	
54							
55							
56							
57							
58							
59							
60							

LOG OF EXPLORATION BORING NO. BH-6

Logged by: **JSO**

Method of Drilling: **8-inch diameter hollow-stem auger** **Date Drilled:** **1/12/2006**

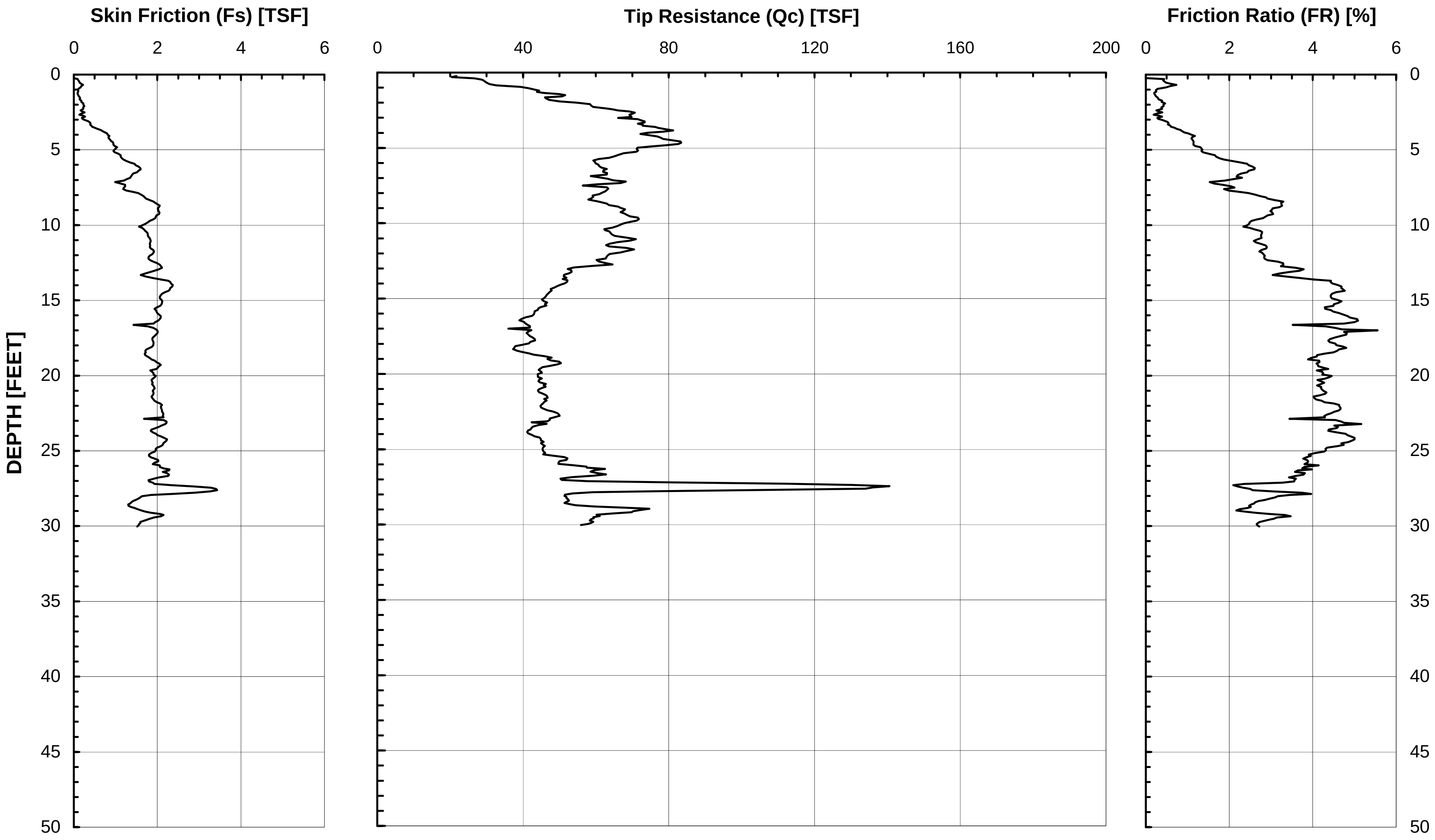
DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
			▨				Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index
1			▨			**<u>LACUSTRINE DEPOSITS:</u>** Sandy lean clay (CL), moderate yellowish brown, medium plasticity, dry to moist.	
2			▨				
3			▨				
4			▨				
5			▨				
	28	CAL		102	13		
6							
7							
8							
9							
10							
						Fat clay (CH), dark yellowish brown, high plasticity, moist, hard.	
	13	SPT					
11							
12							
13							
14							
15							
						Very hard, trace of silt. PP > 5.0 tsf.	
	46	CAL		108	19		
16							
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-6(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/12/2006

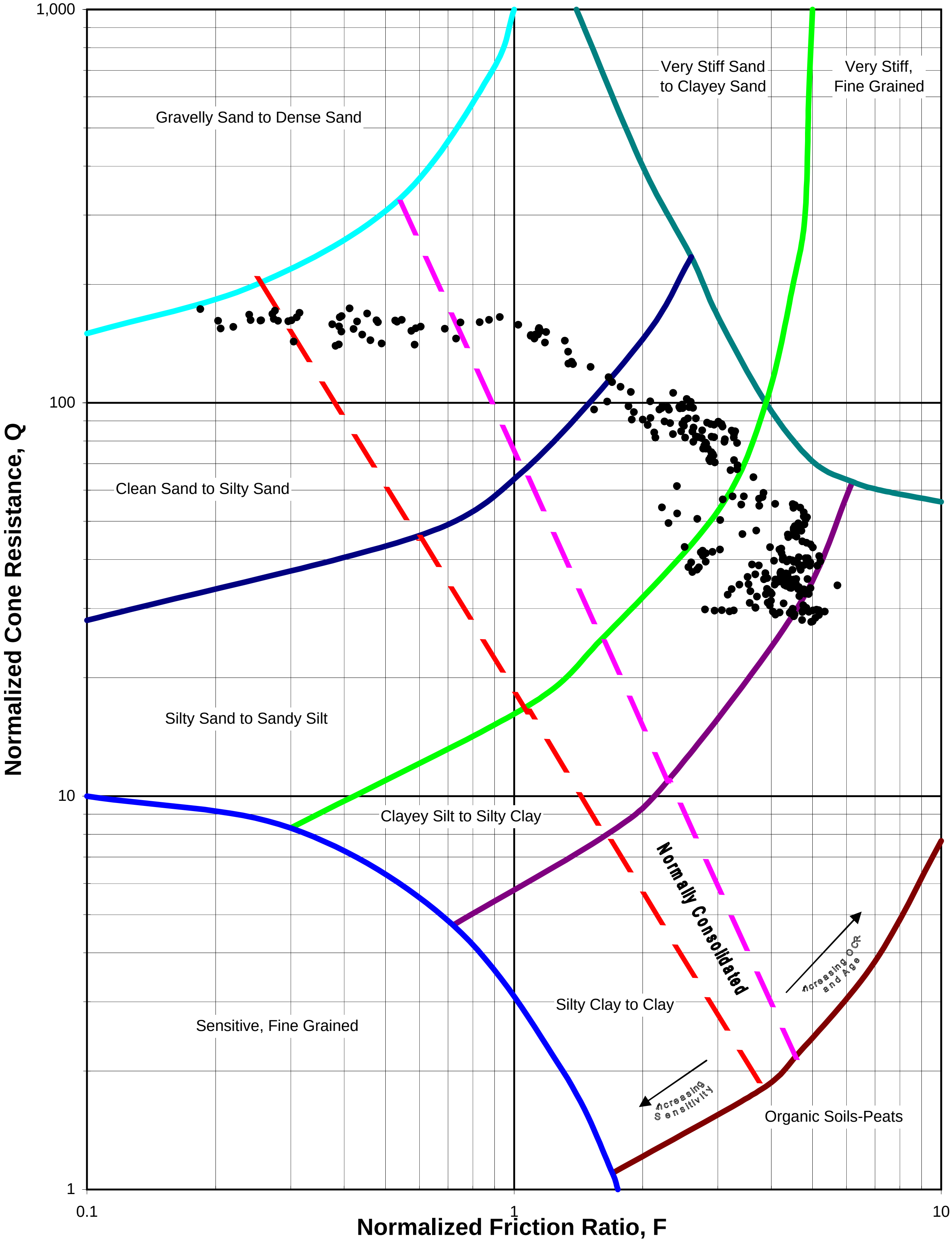
DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	21	SPT				**LACUSTRINE DEPOSITS: (continued)** Interbedded fat clay (CH), dark yellowish brown, high plasticity, moist, hard.	
22							
23							
24							
25						PP > 5.0 tsf.	
26	34	CAL		113	14		
27							
28							
29							
30							
31	19	SPT				PP > 5.0 tsf.	
32						Total depth: 31½ feet	
33						No groundwater encountered	
34							
35							
36							
37							
38							
39							
40							



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-7)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-7.1



Geotechnics Incorporated

SOIL CLASSIFICATION (CPT-7)

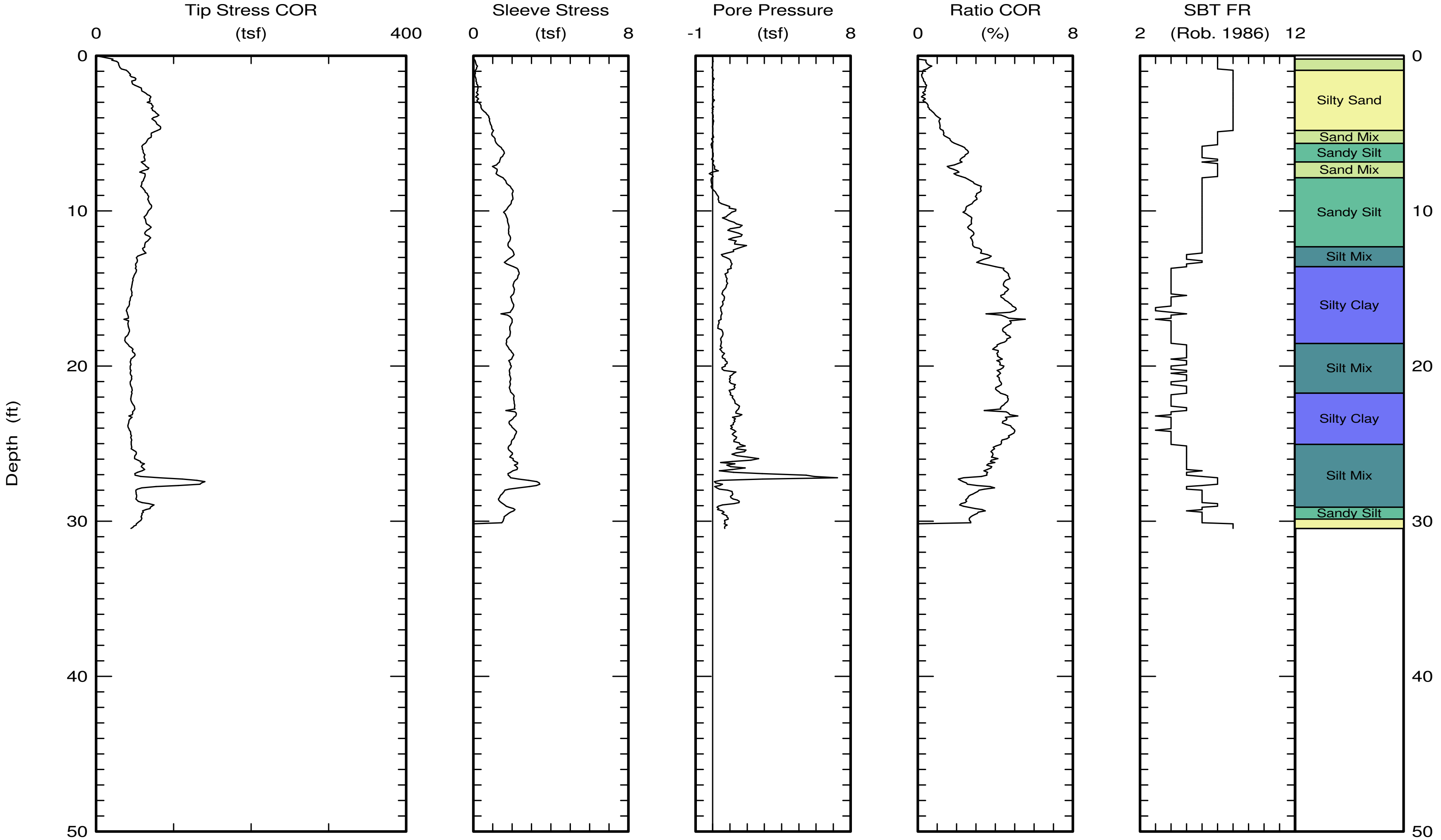
Project No. 0554-075-00
Document No. 06-0015
FIGURE B-7.2

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data
30 ton rig

Date: 17/Jan/2006
Test ID: BH-7
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



Maximum depth: 30.48 (ft)

LOG OF EXPLORATION BORING NO. BH-8

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/17/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
						ALLUVIUM: Well graded sand with silt and gravel (SW-SM), moderate yellowish brown, fine to coarse, dry, very loose.	
1							
2						**LACUSTRINE DEPOSITS:** Lean clay (CL), moderate yellowish brown, medium plasticity, dry to moist.	
3							
4							
5							
	9	SPT				Lean clay to fat clay (CL/CH), light olive gray to dark yellowish brown, medium to high plasticity, moist, hard.	
6							
7							
8							
9							
10							
	44	CAL		108	19		
11							
12							
13							
14							
15							
	10	SPT				Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, thin layers of sandy silt (ML), light olive gray between 5/8" to 1" thick. PP > 4.5 tsf.	
16							
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-8(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/17/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	44	CAL		105	21	**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, trace of sandy silt.	
22							
23							
24							
25							
26	16	SPT				Thin interbedded layers of sandy silt (ML), moderate yellow brown, fine, dry to moist.	
27							
28							
29							
30				109	19		
31	48	CAL					
32						Total depth: 31 feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							

LOG OF EXPLORATION BORING NO. BH-9

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/16/2006

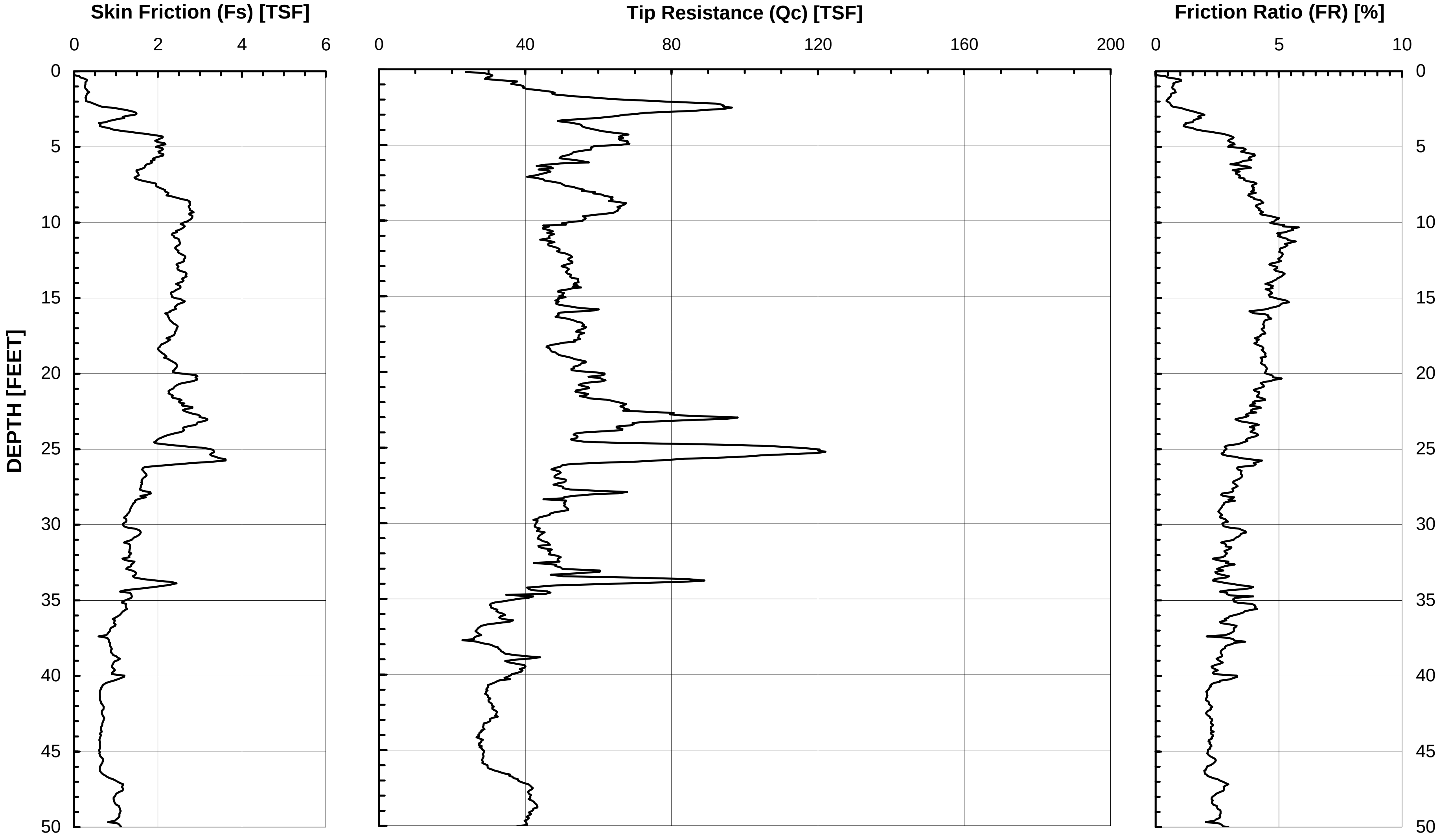
DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
			▨			**ALLUVIUM:** Well graded sand with silt and gravel (SW-SM), moderate yellowish brown, fine to coarse, dry, very loose.	Gradation Hydrometer Atterberg Limits Maximum Density Optimum Moisture Expansion Index R-Value
1			▨				
2			▨			**LACUSTRINE DEPOSITS:** Lean clay to fat clay (CL/CH), dark yellowish brown, dry to moist, hard, trace of sandy silt (ML), light olive gray.	
3			▨				
4			▨				
5							
6	12	SPT					
7							
8							
9							
10				112	18	Fat clay (CH), dark yellowish brown, high plasticity, moist, trace of sandy silt.	Consolidation
11	50	CAL					
12							
13							
14							
15							
16	14	SPT				At 16 feet- interbedded silty sand to sandy silt (SM/ML), light olive gray, approximately 1" thick.	
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-9(continued)

Logged by: **JSO**

Method of Drilling: **8-inch diameter hollow-stem auger** **Date Drilled:** **1/16/2006**

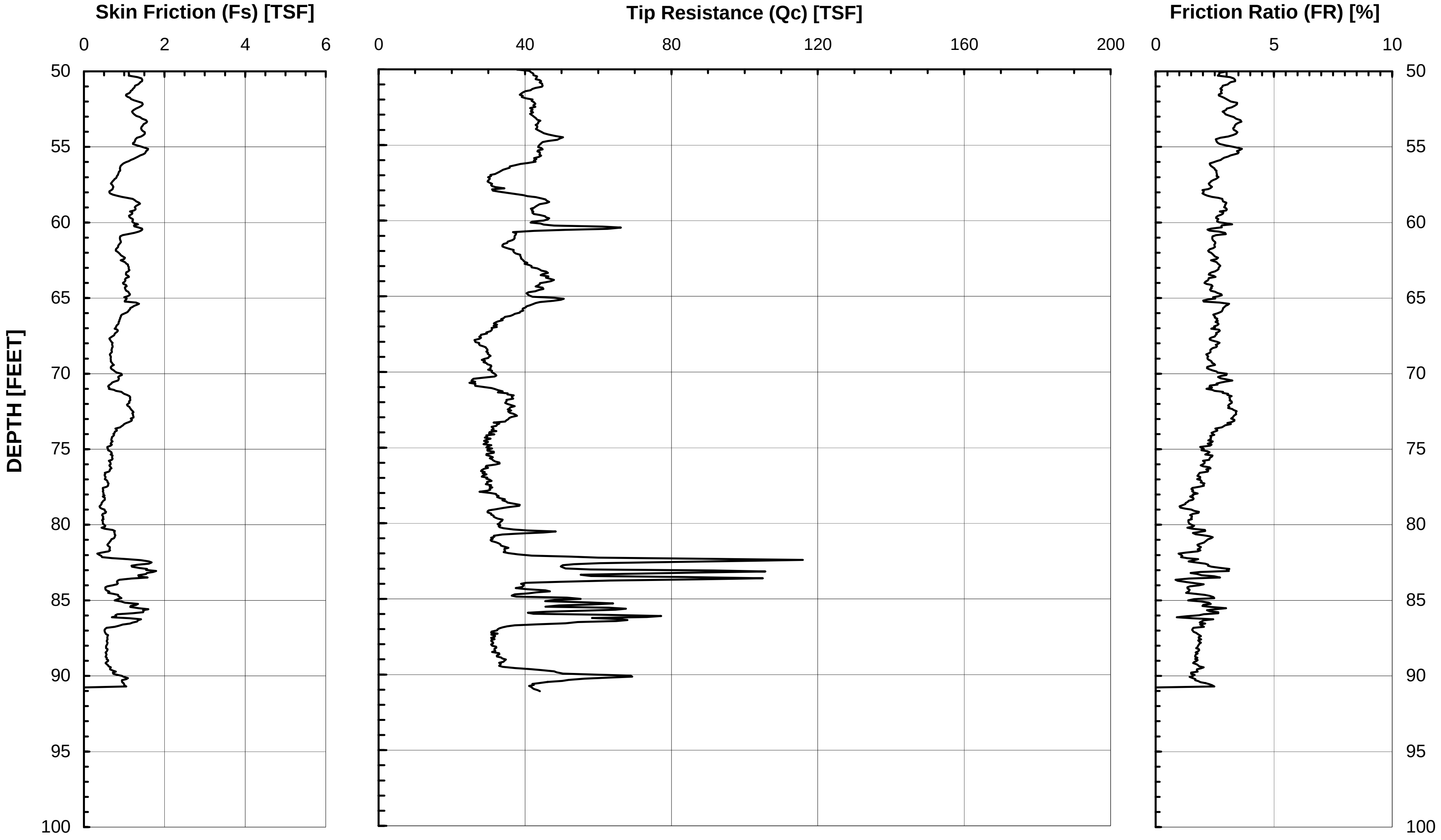
DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	54	CAL		109	20	**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, thin layers of sandy silt, approximately 1" thick.	Consolidation
22							
23							
24							
25							
26	13	SPT					
27							
28							
29							
30							
31	39	CAL		102	23		
32						Total depth: 31 feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-10)

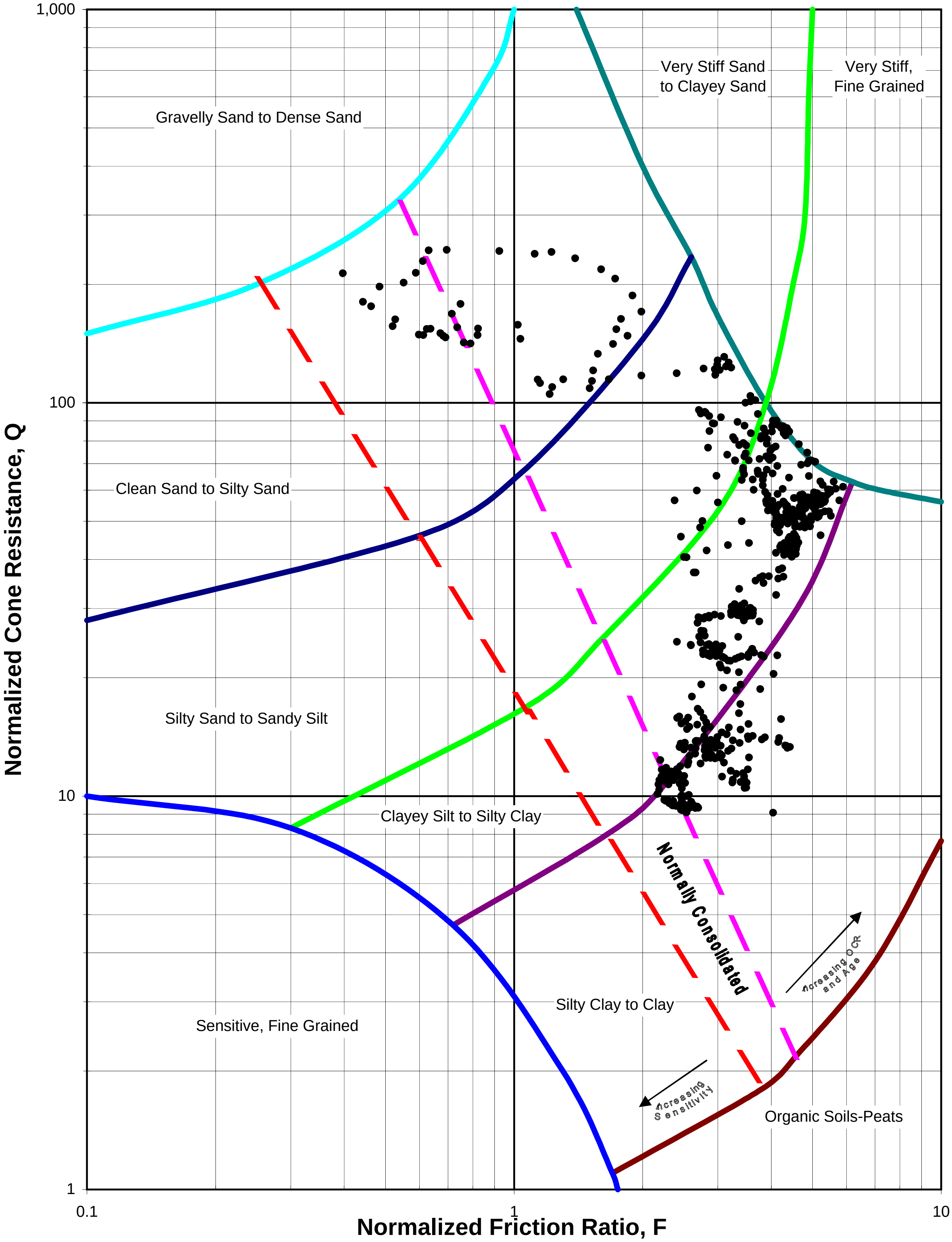
Project No. 0554-075-00
Document No. 06-0015
FIGURE B-10.1



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-10)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-10.2



Geotechnics
Incorporated

SOIL CLASSIFICATION (CPT-10)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-10.3

Geotechnics - Niland Power Substation
Niland, CA

BH-10
Shear Wave Measurements

Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
5.25	7.25	8.52	850.94	
10.31	11.46	20.63	555.43	347.52
15.31	16.11	30.38	530.14	476.65
20.34	20.95	37.84	553.53	648.76
25.22	25.71	46.44	553.64	554.11
30.53	30.94	54.87	563.82	619.91
35.53	35.88	62.88	570.61	617.15
40.25	40.56	70.20	577.77	639.25
45.65	45.92	78.08	588.15	680.66
50.63	50.88	85.98	591.72	627.00
55.14	55.37	94.09	588.44	553.63
60.34	60.55	101.27	597.88	721.53
65.57	65.76	107.36	612.52	856.08
70.38	70.56	113.45	621.92	787.69
75.09	75.26	118.67	634.16	900.17
80.44	80.60	124.93	645.12	852.87
85.46	85.61	130.43	656.34	911.07
90.36	90.50	136.88	661.15	758.46

Shear Wave Source Offset = 5 ft

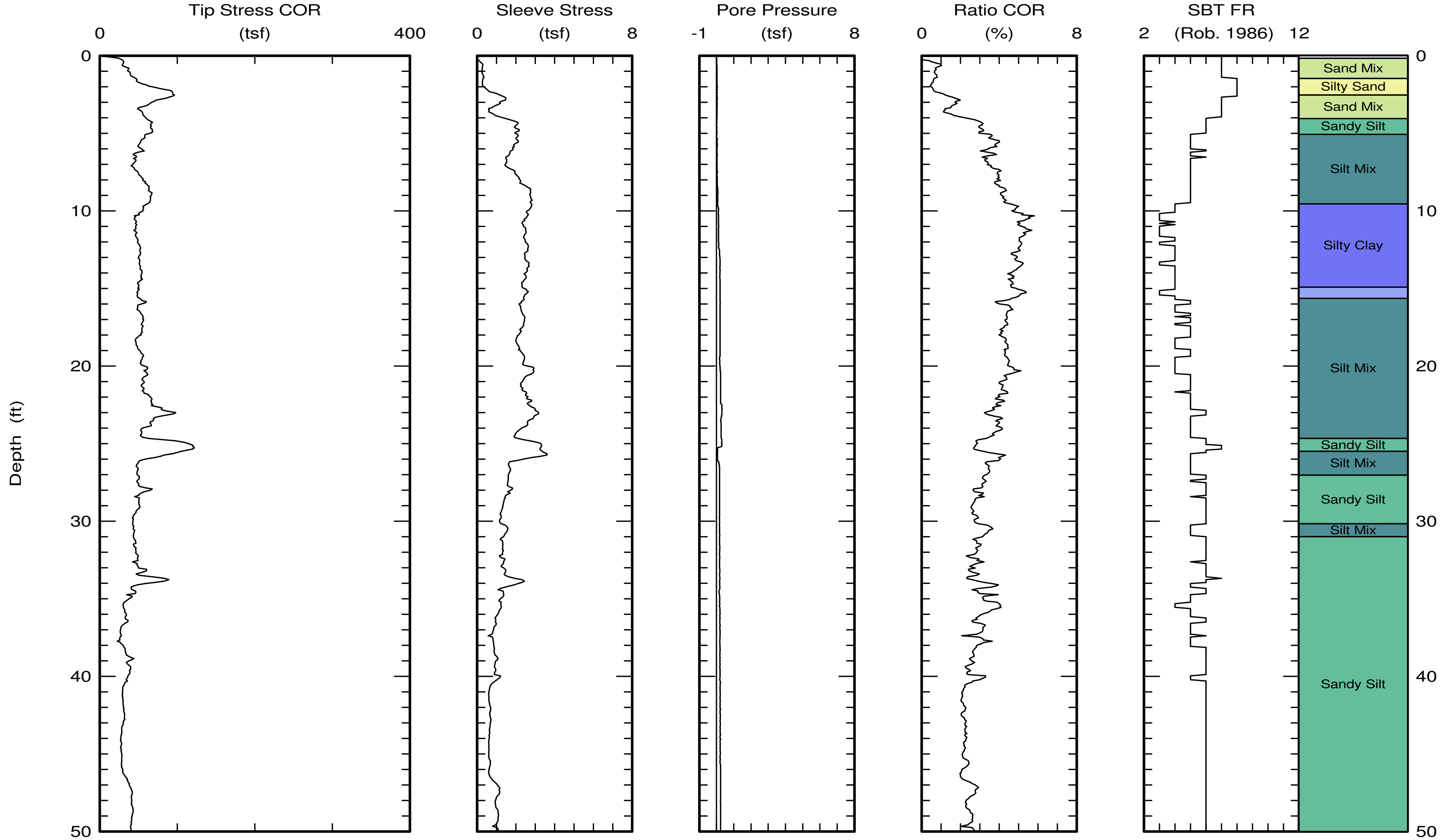
S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data
30 ton rig

Date: 16/Jan/2006
Test ID: BH-10
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



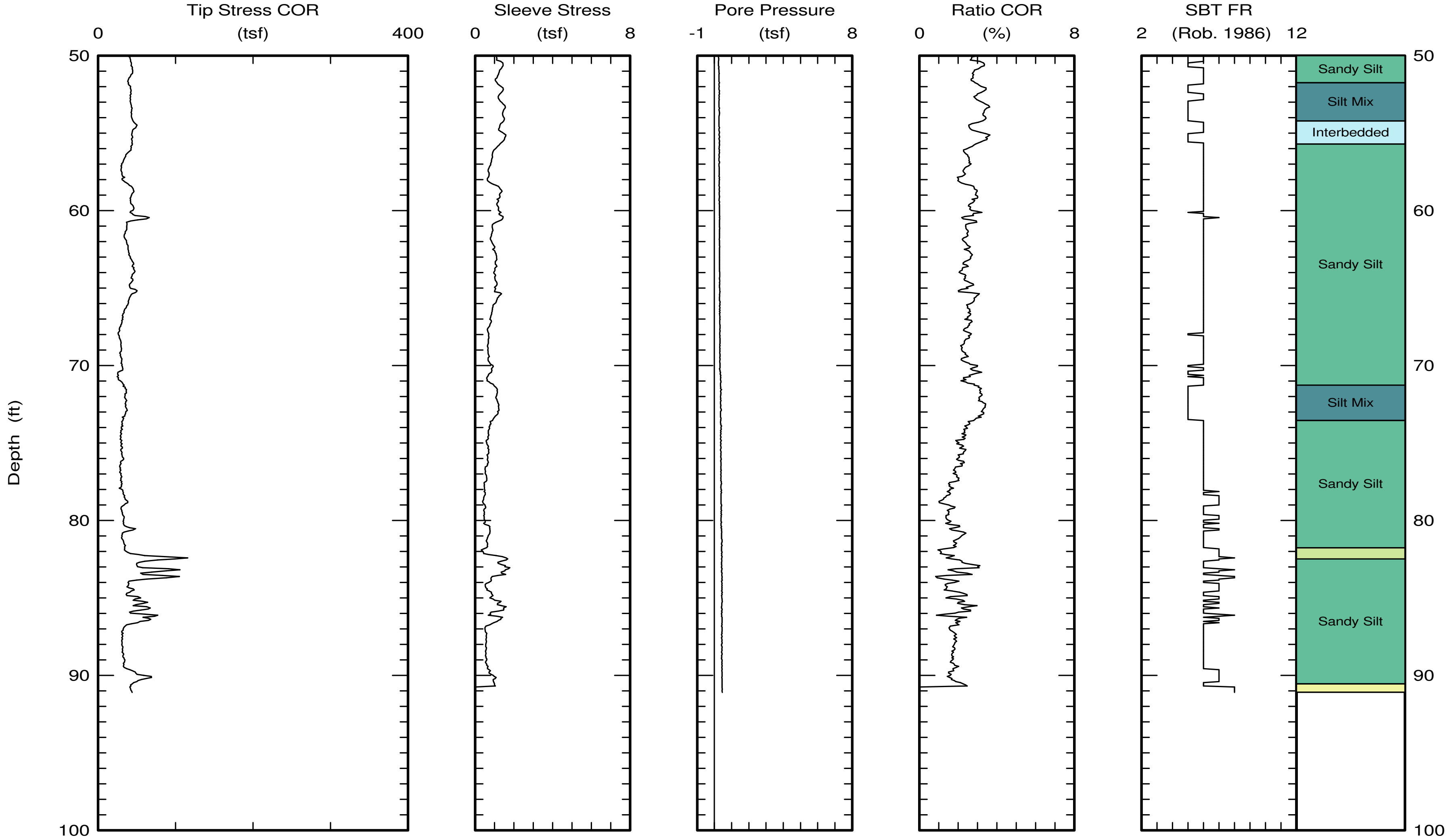
Maximum depth: 91.10 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

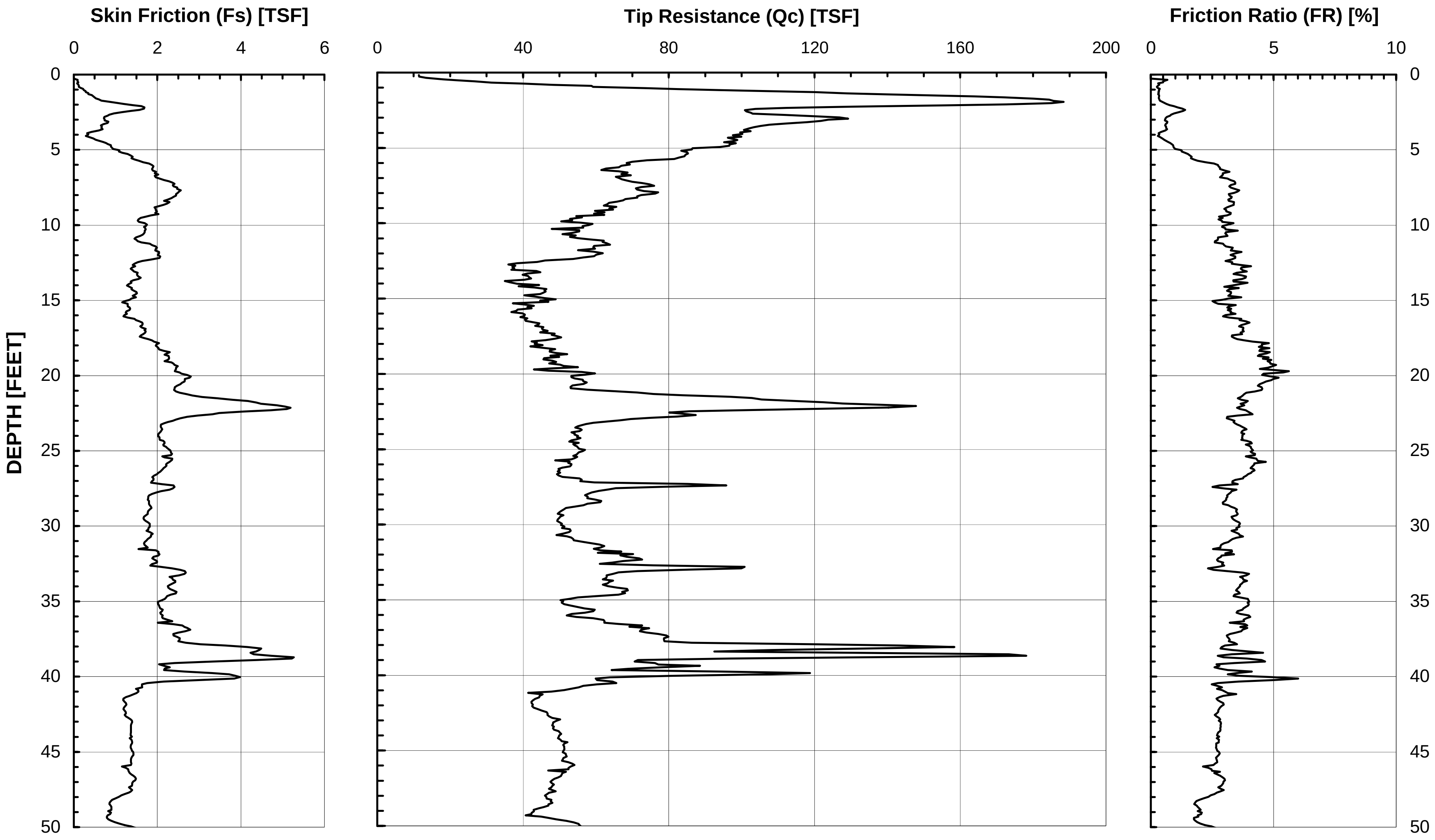
CPT Data
30 ton rig

Date: 16/Jan/2006
Test ID: BH-10
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



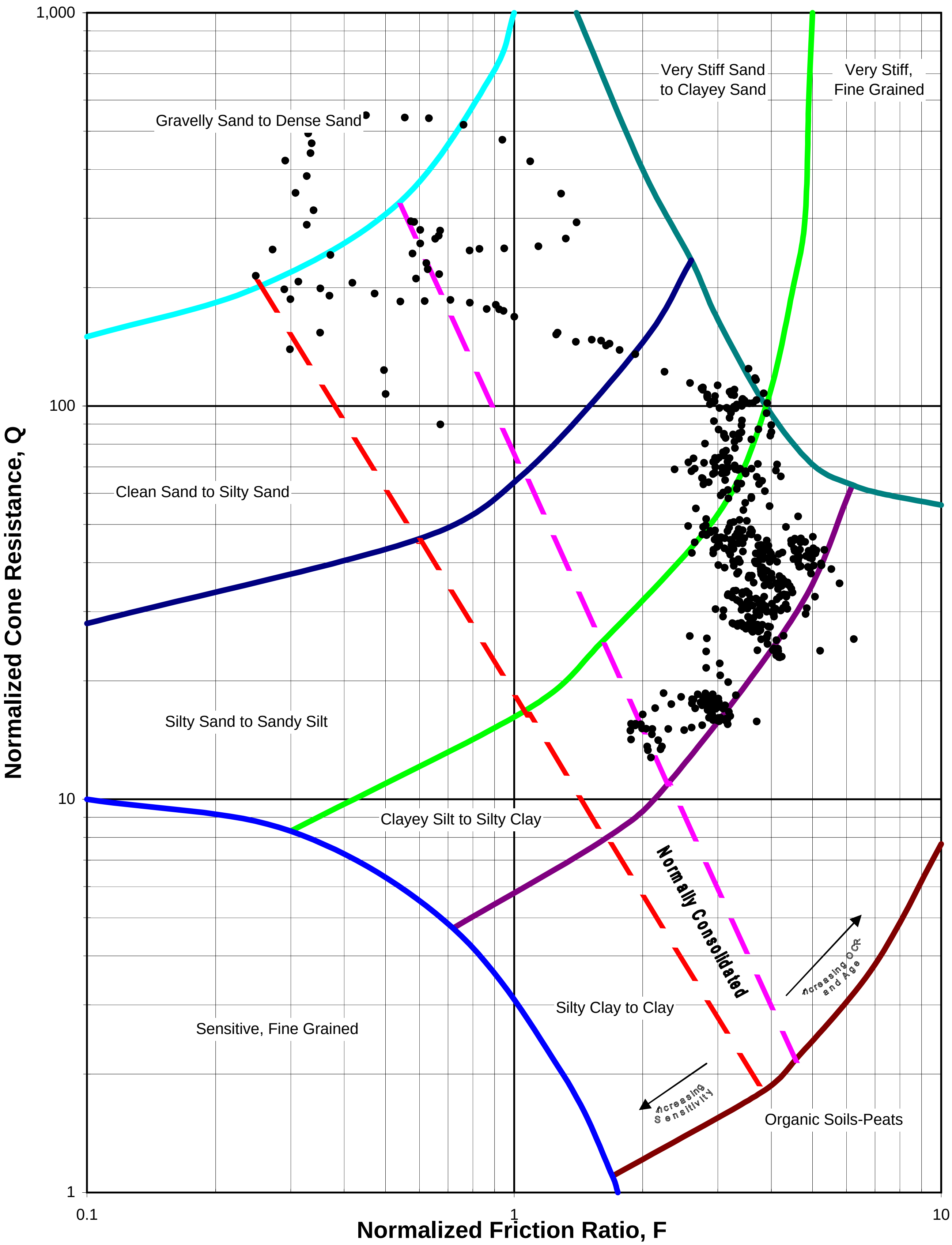
Maximum depth: 91.10 (ft)
Page 2 of 2



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-11)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-11.1



Geotechnics Incorporated

SOIL CLASSIFICATION (CPT-11)

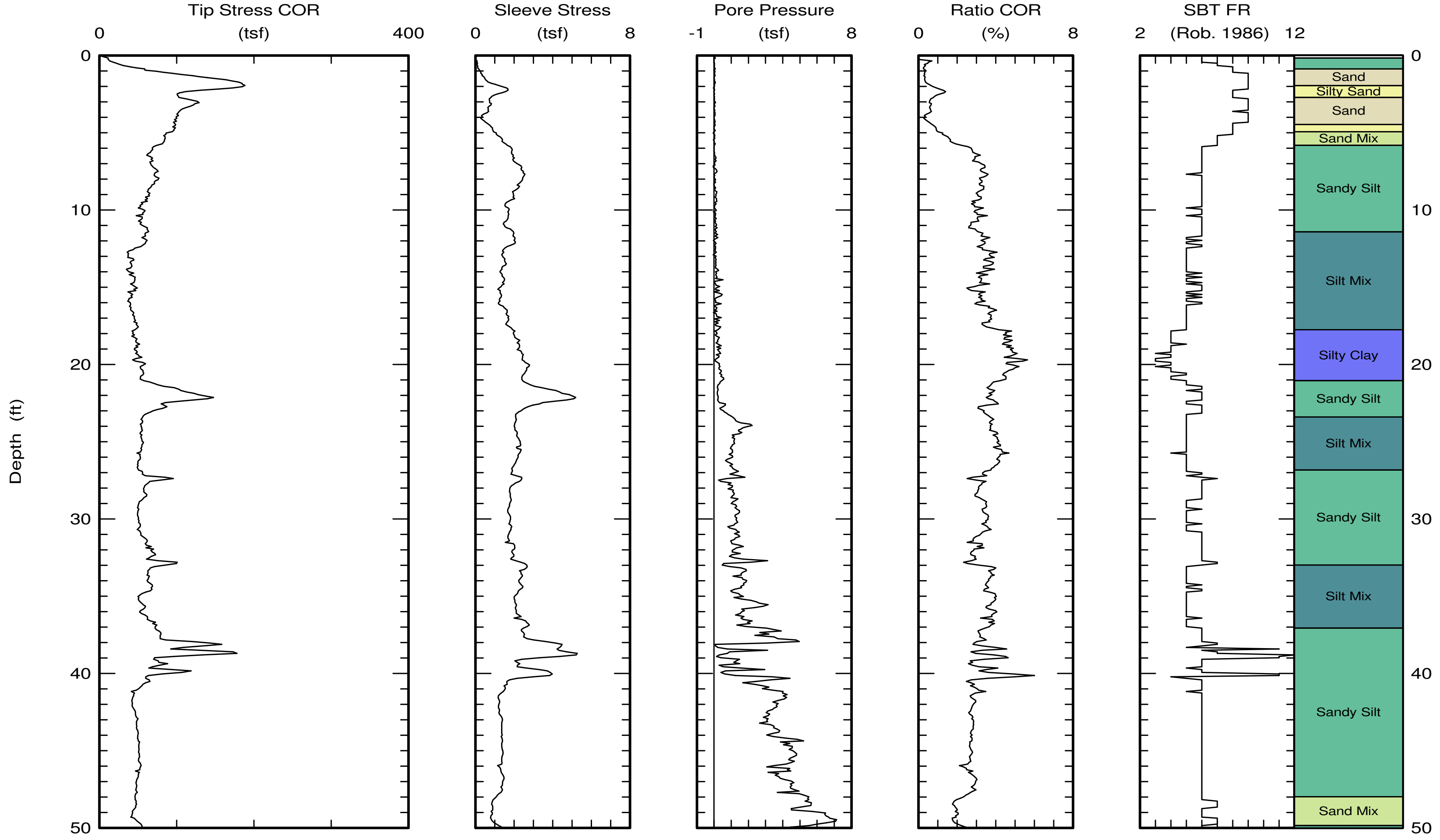
Project No. 0554-075-00
Document No. 06-0015
FIGURE B-11.2

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data
30 ton rig

Date: 17/Jan/2006
Test ID: BH-11
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



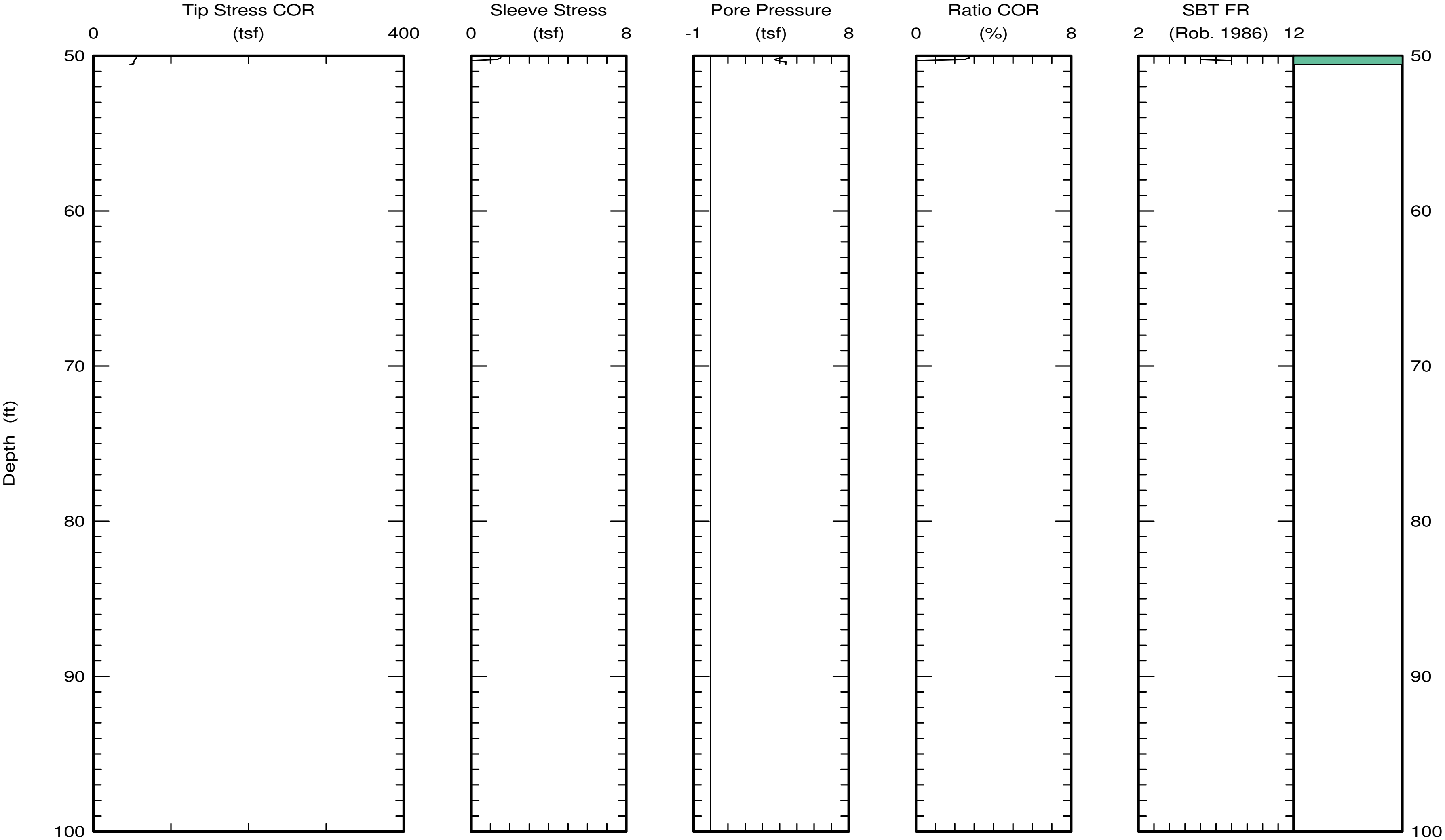
Maximum depth: 50.59 (ft)
Page 1 of 2

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data
30 ton rig

Date: 17/Jan/2006
Test ID: BH-11
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



Maximum depth: 50.59 (ft)
Page 2 of 2

LOG OF EXPLORATION BORING NO. BH-12

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/17/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
			▨			**ALLUVIUM:** Well graded sand with silt (SW-SM), moderate yellowish brown, fine to coarse, dry, very loose, trace of gravel..	Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index
1			▨			**LACUSTRINE DEPOSITS:** Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, hard, trace of sandy silt to silty sand.	
2			▨				
3			▨				
4			▨				
5	21	CAL		106	14		Gradation Direct Shear
6							
7							
8							
9							
10	7	SPT					
11							
12							
13							
14							
15	30	CAL		106	19	Thin interbedded layers of lean clay (CL), dark yellowish brown, medium plasticity, moist, hard, and sandy silt (ML), light olive gray, fine, dry to moist.	Gradation Direct Shear
16							
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-12(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/17/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	11	SPT				**<u>LACUSTRINE DEPOSITS: (continued)</u>** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, thin layers of silty sand.	
22							
23							
24							
25							
26	39	CAL		106	20		Gradation Direct Shear
27							
28							
29							
30							
31	18	SPT				Iron oxide staining.	
32						Total depth: 31½ feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							

PROJECT NO. 0554-075-00 **GEOTECHNICS INCORPORATED** **FIGURE B-12.2**

LOG OF EXPLORATION BORING NO. BH-13

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/17/2006

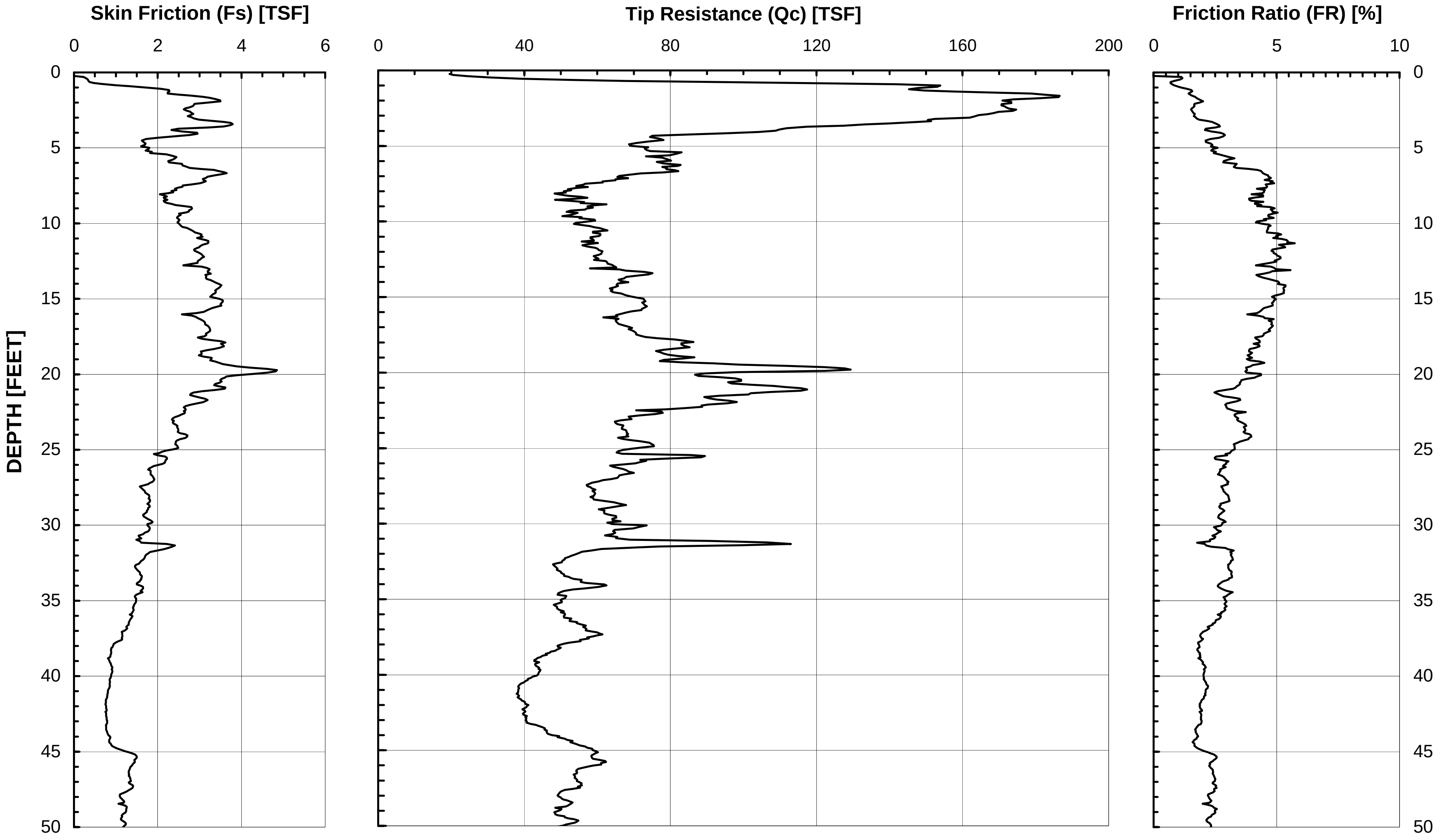
DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
						ALLUVIUM: Well graded sand with silt (SW-SM), moderate yellowish brown, fine to coarse, dry, loose, trace of gravel.	
1							
2						**LACUSTRINE DEPOSITS:** Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, hard, trace of sandy silt.	
3							
4							
5							
6	9	SPT					
7							
8							
9							
10							
11	31	CAL				Thin interbedded layers of sandy silt (ML), moderate yellowish brown to light olive gray between 5/8" to 1" thick.	Gradation Direct Shear
12							
13							
14							
15							
16	13	SPT					
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-13(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/17/2006

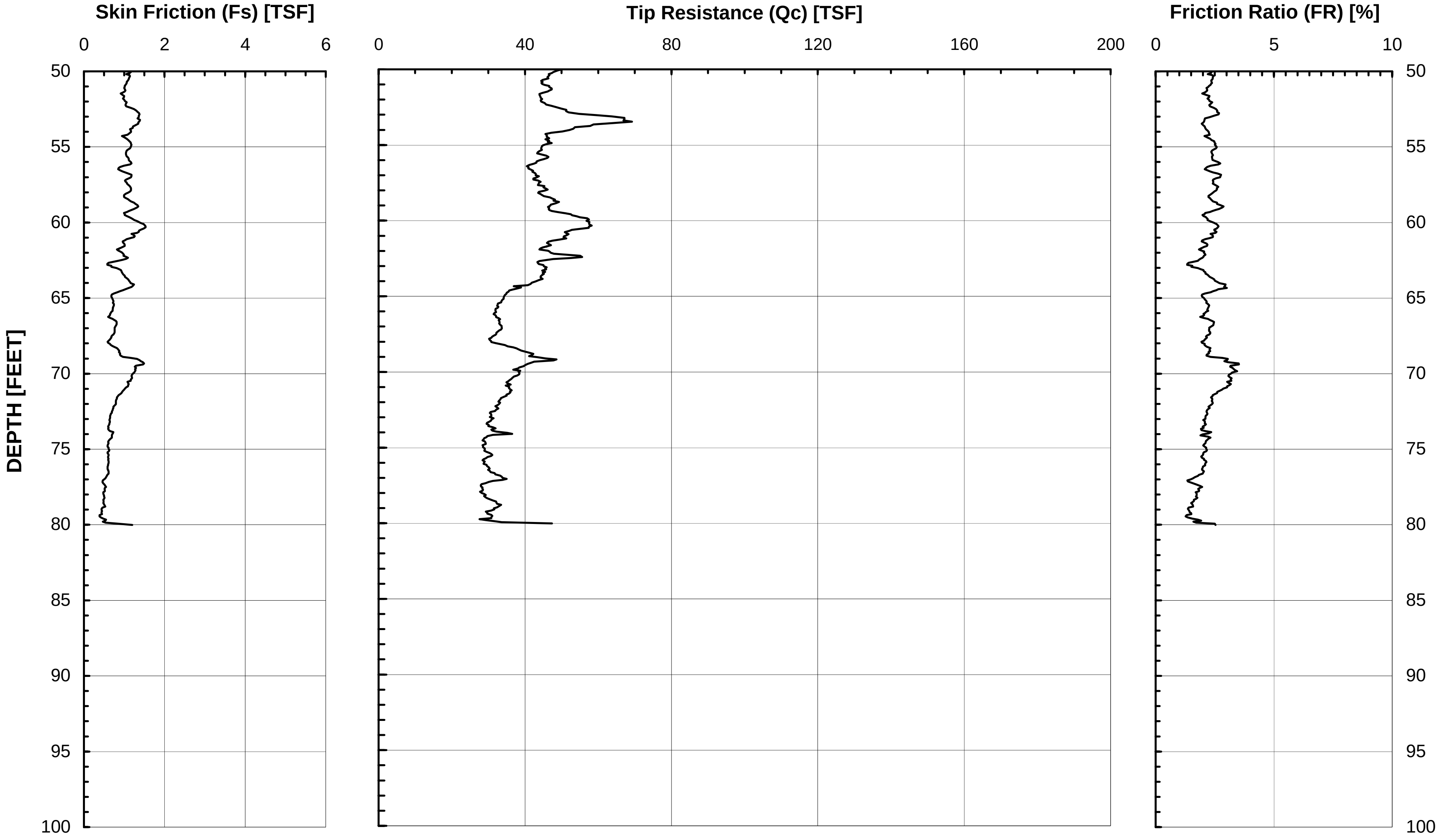
DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	46	CAL		111	18	**LACUSTRINE DEPOSITS: (continued)** Lean clay to fat clay (CL/CH), dark yellowish brown, medium to high plasticity, moist, hard, thin interbedded layers of sandy silt.	Gradation Direct Shear
22							
23							
24							
25							
26	16	SPT				Micaceous between 25-25½ feet.	
27							
28							
29							
30							
31	41	CAL		110	18		Gradation Direct Shear
32						Total depth: 31 feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-14)

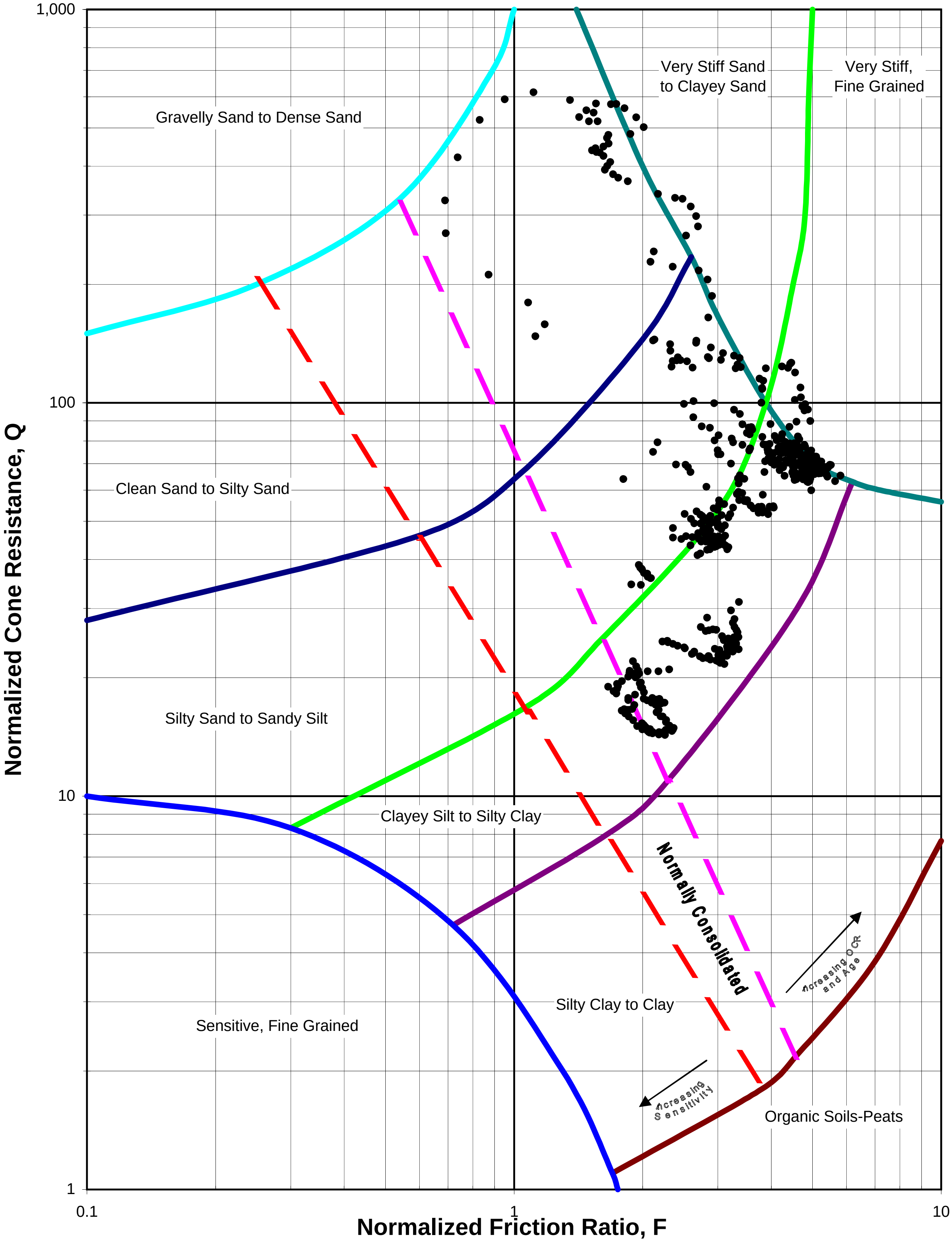
Project No. 0554-075-00
Document No. 06-0015
FIGURE B-14.1



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-14)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-14.2



Geotechnics Incorporated

SOIL CLASSIFICATION (CPT-14)

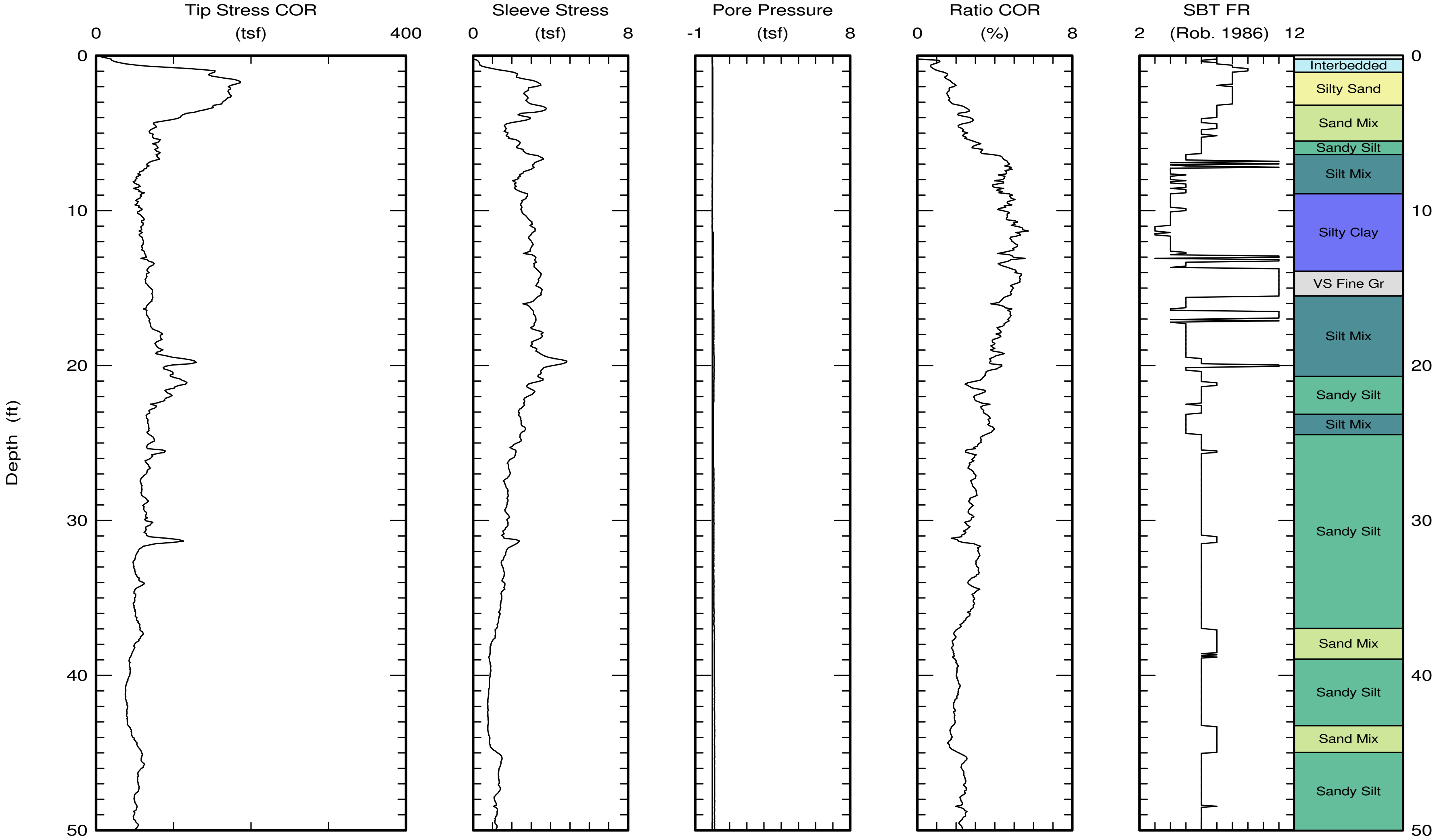
Project No. 0554-075-00
Document No. 06-0015
FIGURE B-14.3

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data
30 ton rig

Date: 16/Jan/2006
Test ID: BH-14
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



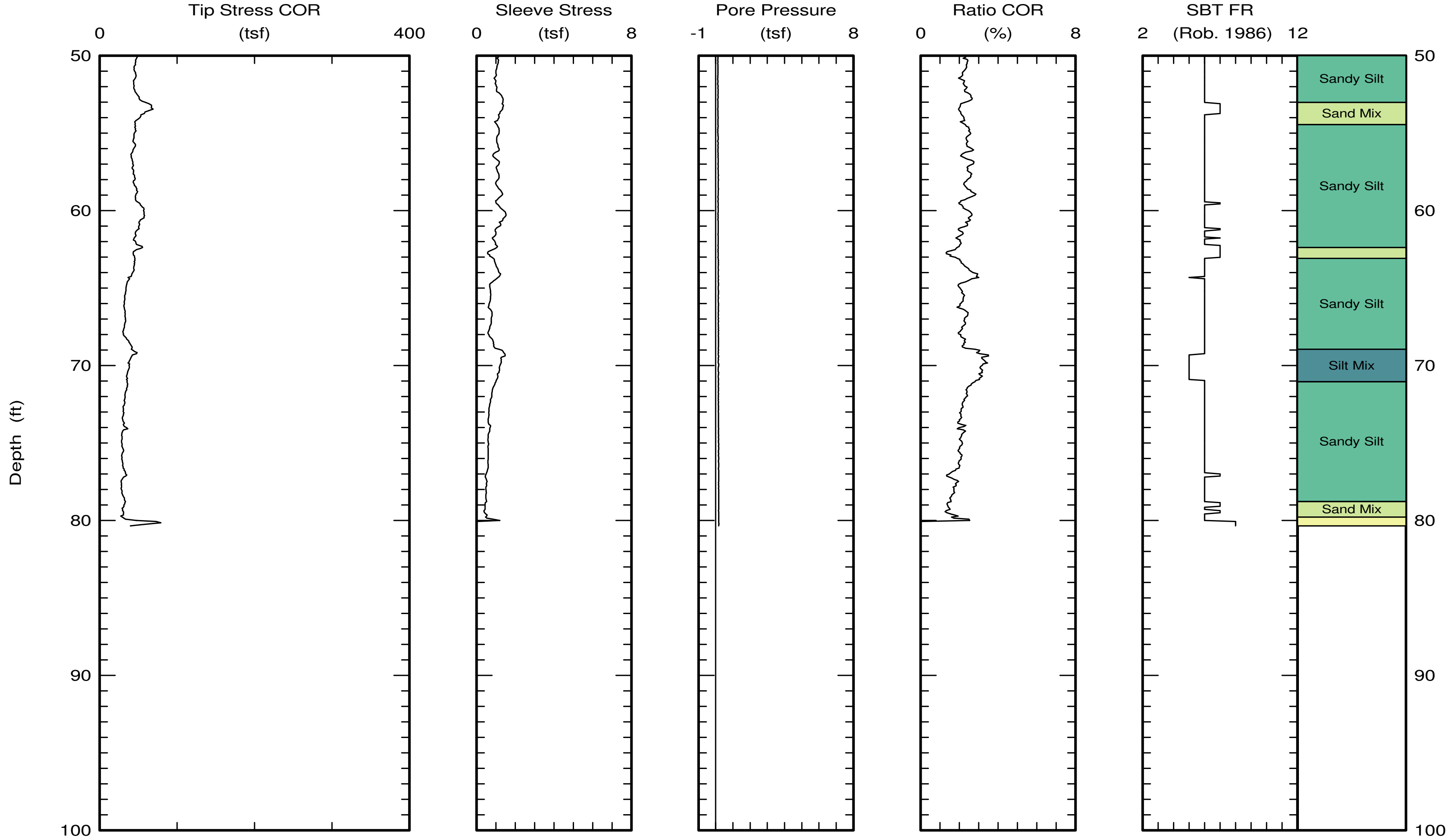
Maximum depth: 80.36 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

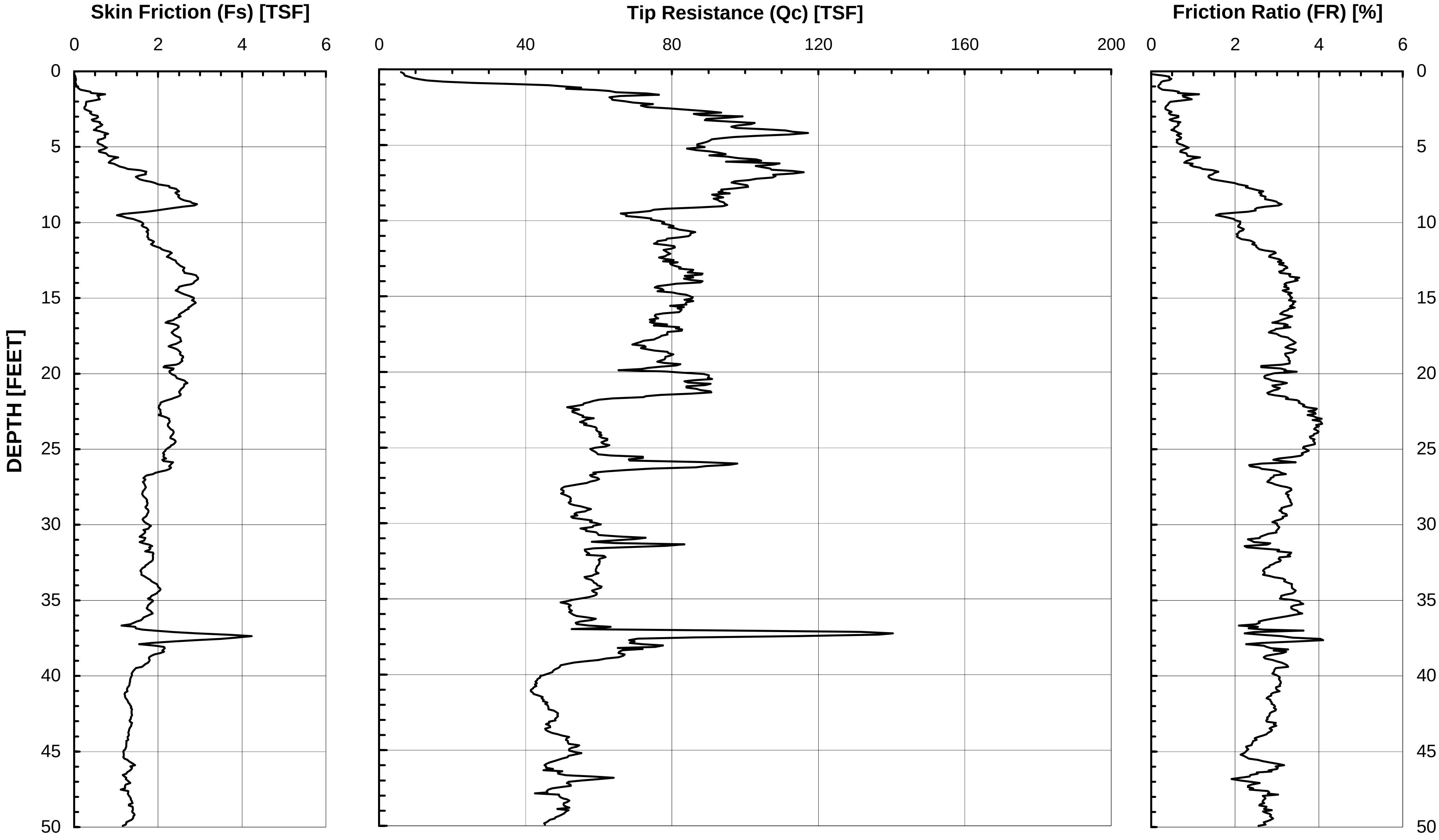
CPT Data
30 ton rig

Date: 16/Jan/2006
Test ID: BH-14
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



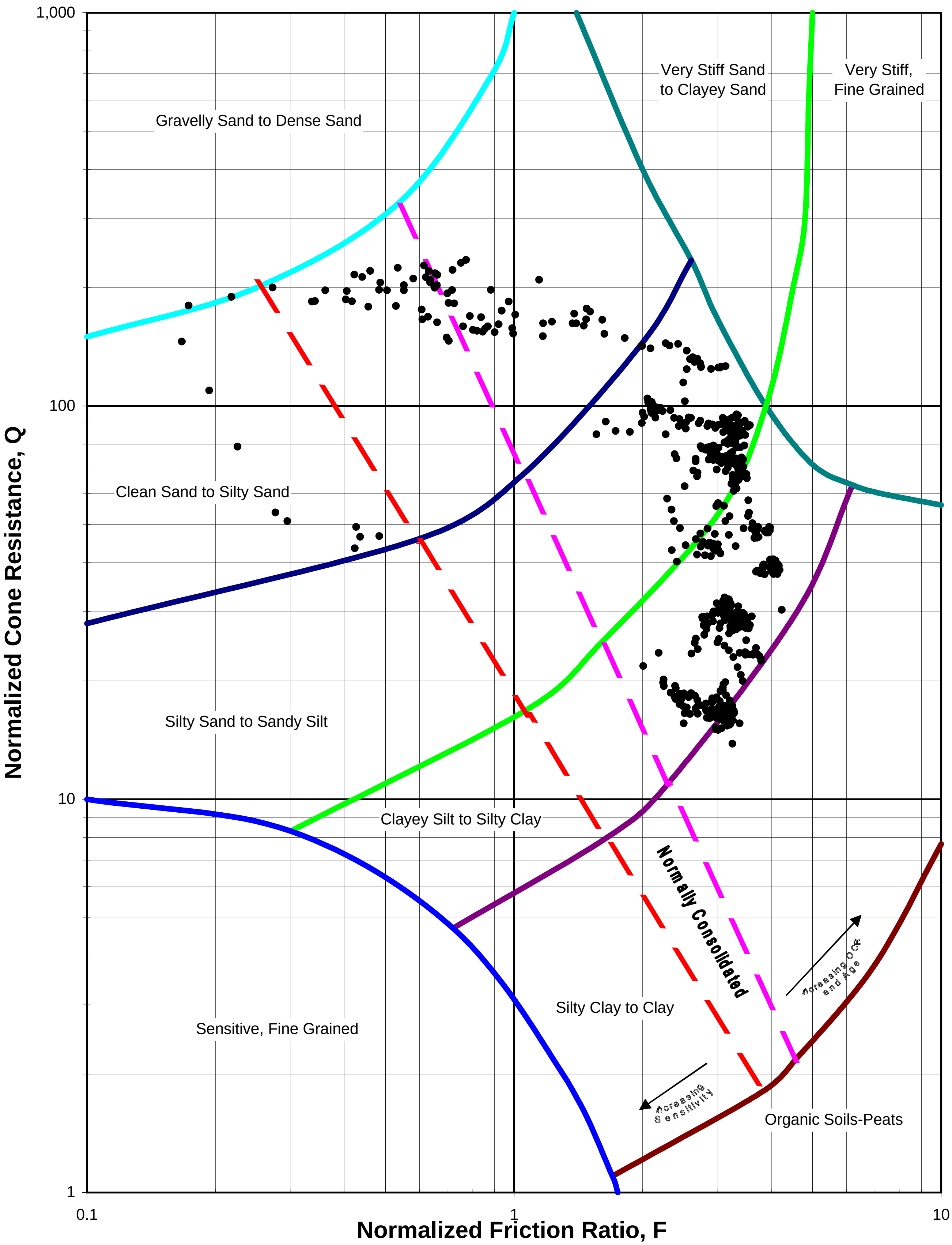
Maximum depth: 80.36 (ft)
Page 2 of 2



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-15)

Project No. 0554-075-00
Document No. 06-0015
FIGURE B-15.1



Geotechnics Incorporated

SOIL CLASSIFICATION (CPT-15)

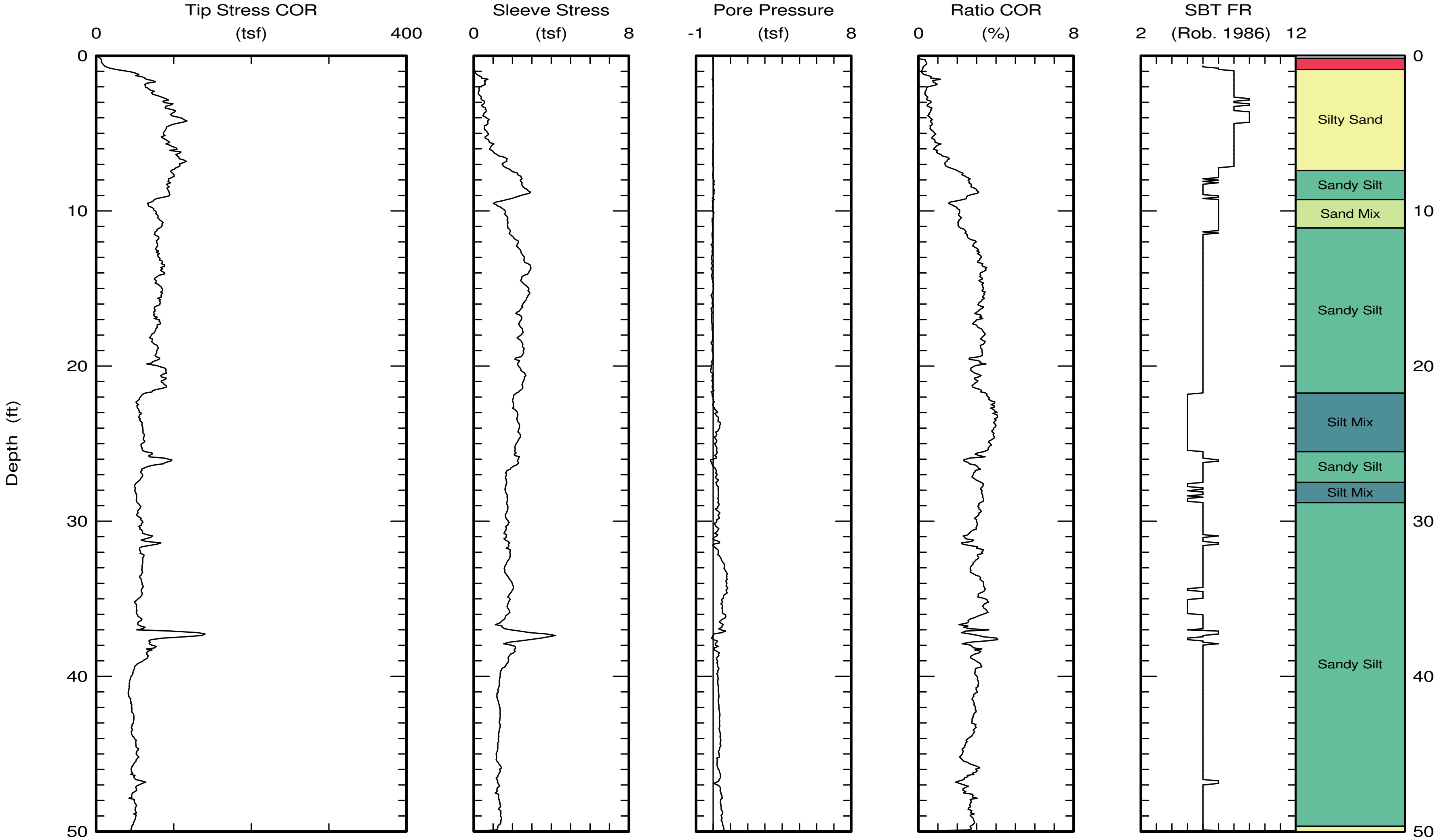
Project No. 0554-075-00
Document No. 06-0015
FIGURE B-15.2

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
skehoe@msn.com

CPT Data
30 ton rig

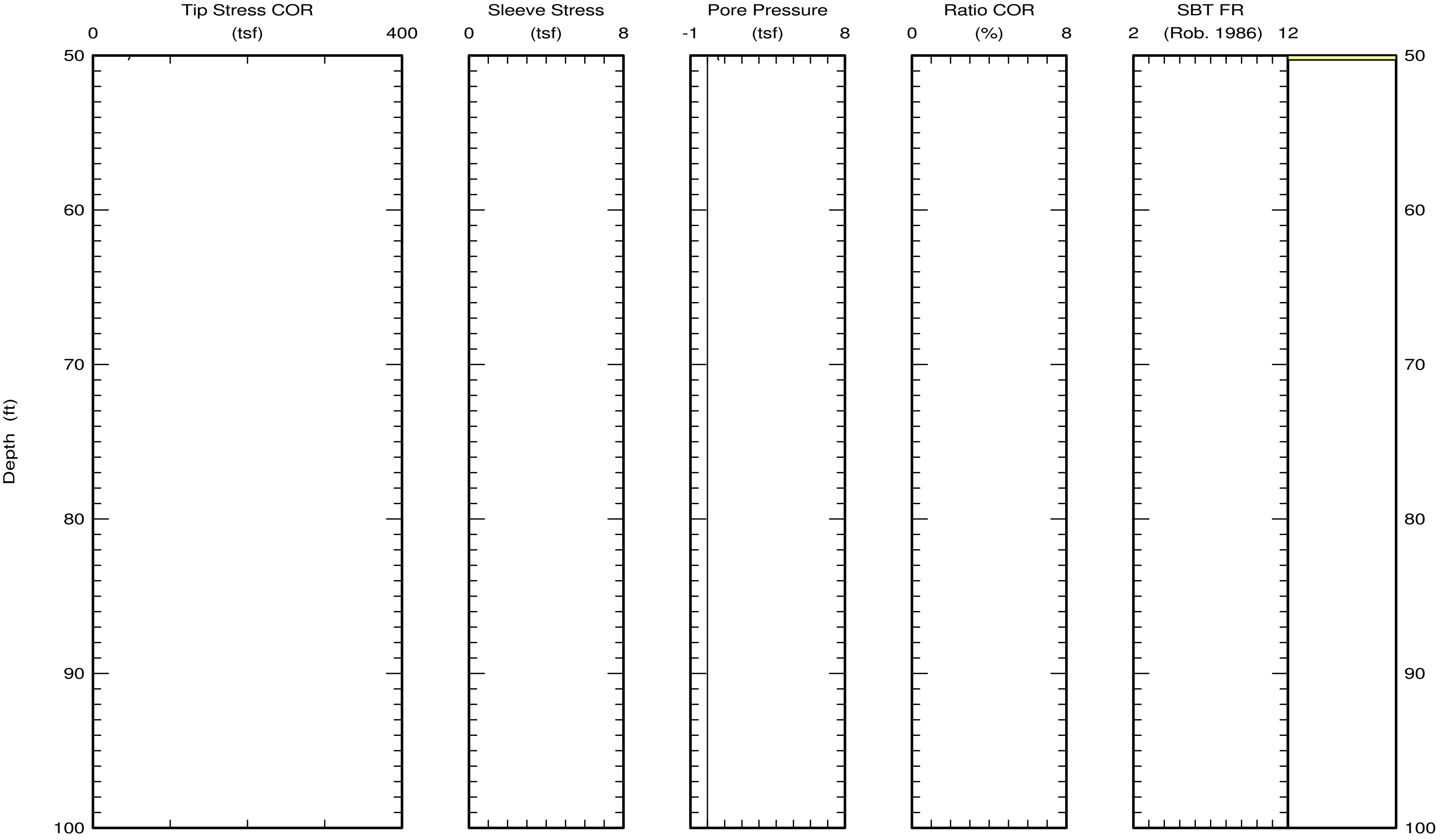
Date: 17/Jan/2006
Test ID: BH-15
Project: Niland

Client: Geotechnics Inc
Job Site: Niland Power Substation



Maximum depth: 50.29 (ft)
Page 1 of 2

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 skehoe@msn.com	CPT Data 30 ton rig	Date: 17/Jan/2006 Test ID: BH-15 Project: Niland
	Client: Geotechnics Inc Job Site: Niland Power Substation	



Maximum depth: 50.29 (ft)

Page 2 of 2

LOG OF EXPLORATION BORING NO. BH-16

Logged by: JSO
Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/16/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
						ALLUVIUM: Well graded sand with silt (SW-SM), moderate yellowish brown, fine to coarse, dry, very loose, trace of gravel.	
1							
						LACUSTRINE DEPOSITS: Fat clay (CH), dark yellowish brown, high plasticity, dry to moist, hard, thin interbedded layers of silty sand to sandy silt.	
2							
3							
4							
5							
	34	CAL		107	14		
6							
7							
8							
9							
10							
	12	SPT				Trace of sandy silt (ML), light olive gray, fine, dry to moist.	
11							
12							
13							
14							
15							
	59	CAL		110	18		
16							
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-16(continued)

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/16/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	15	SPT				**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, thin layers of sandy silt, approximately 5/8" thick.	
22							
23							
24							
25						Trace of sandy silt (ML), moderate yellowish brown.	
26	40	CAL		108	20		
27							
28							
29							
30						Thin interbedded layers of sandy silt (ML), moderate yellowish brown, approximately 1" thick.	
31	15	SPT					
32						Total depth: 31½ feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							

LOG OF EXPLORATION BORING NO. BH-17

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/16/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
			▨			**ALLUVIUM:** Well graded sand with silt (SW-SM), moderate yellowish brown, fine to coarse, dry, very loose, trace gravel.	Gradation Hydrometer Atterberg Limits Soluble Sulfate Soluble Chloride pH & Resistivity Expansion Index
1			▨				
2			▨			**LACUSTRINE DEPOSITS:** Fat clay (CH), dark yellowish brown, high plasticity, dry to moist, hard, thin layers of sandy silt.	
3			▨				
4			▨				
5							
6	9	SPT					
7							
8							
9							
10							
11	37	CAL		106	20	Sandy silt to silty sand (ML/SM), light olive gray, dry to moist, fine, medium dense.	
12							
13						Fat clay (CH), dark yellowish brown, high plasticity, moist, hard.	
14							
15							
16	12	SPT					
17							
18							
19							
20							

LOG OF EXPLORATION BORING NO. BH-17(continued)

Logged by: **JSO**

Method of Drilling: **8-inch diameter hollow-stem auger** **Date Drilled:** **1/16/2006**

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
	43	CAL		110	18	**LACUSTRINE DEPOSITS: (continued)** fat clay (CH), dark yellowish brown, high plasticity, moist, hard.	
21							
22							
23							
24							
25						Thinly bedded layer of of sandy silt (ML), moderate yellowish brown, iron oxide staining.	
26	14	SPT					
27							
28							
29							
30				106	22	Trace of sandy silt (ML), moderate yellowish brown.	
	34	CAL					
31							
32						Total depth: 31 feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							

LOG OF EXPLORATION BORING NO. BH-18

Logged by: JSO

Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/16/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
						ALLUVIUM: Well graded sand with silt (SW-SM), moderate yellowish brown, fine to coarse, dry, loose, trace of gravel.	
1							
2						**LACUSTRINE DEPOSITS:** Fat clay (CH), dark yellowish brown, high plasticity, dry to moist, hard, trace of sandy silt.	
3							
4							
5							
6	18	CAL		106	16		
7							
8							
9							
10						Thin layers of sandy silt (ML), pale yellowish brown to dark yellowish orange.	
11	11	SPT					
12							
13							
14							
15						Moist.	
16	61	CAL		111	14		
17							
18							
19							
20							

PROJECT NO. 0554-075-00 **GEOTECHNICS INCORPORATED** **FIGURE B-18.1**

LOG OF EXPLORATION BORING NO. BH-18(continued)

Logged by: JSO
Method of Drilling: 8-inch diameter hollow-stem auger **Date Drilled:** 1/16/2006

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
21	12	SPT				**LACUSTRINE DEPOSITS: (continued)** Fat clay (CH), dark yellowish brown, high plasticity, moist, hard, thin interbedded layers of sandy silt (ML).	
22							
23							
24							
25						Trace of sandy silt (ML), moderate yellowish brown.	
26	52	CAL		109	20		
27							
28							
29							
30						Thin interbedded layers of sandy silt (ML), moderate yellowish brown approximately 1" thick.	
31	9	SPT					
32						Total depth: 31½ feet No groundwater encountered	
33							
34							
35							
36							
37							
38							
39							
40							

PROJECT NO. 0554-075-00 **GEOTECHNICS INCORPORATED** **FIGURE B-18.2**

APPENDIX C

FIELD PERCOLATION TESTING

In-situ percolation testing was conducted at three locations within the proposed storm water detention basins. The percolation tests were conducted between January 17 and 20 of this year in general accordance with the *Imperial County Uniform Policy and Method for Soils Evaluation, Testing and Reporting.* The field percolation test data is presented in Figures C-1 through C-3. The approximate locations of the percolation tests are shown on the Exploration Plan, Figure 2.

The test holes were drilled to a depth of 5 feet below the existing ground surfaces using a truck mounted 8-inch diameter hollow stem auger drill rig. The test holes were prepared by first removing all loose soils and debris, and then placing 12 inches of pea gravel in the bottom of the hole. The holes were then presoaked by filling them with clear water to 6 inches above the pea gravel. The columns of water were re-established periodically during the presoaking period (which lasted a total of 24 hours). After 24 hours, water level readings were taken relative to a fixed reference point at 60 minute intervals for 5 hours. The stabilized rate of water drop (t) was reached when two successive determinations did not vary by more than 10 percent. Based on the test results, the percolation rate of the in-situ soils ranges from approximately 0 to ¼ gallon per square foot per day at the test depth.

Test Hole No. PT-1 Date Excavated 1/17/2006
Depth of Test Hole 5 feet Earth Material Clay/Fat clay CL/CH
Actual Percolation Tested by JSO Date 1/20/2006 Test Hole Diameter 8 in.
Case Number

Reference: Imperial County Public Health Department, Division of Environmental Health Services,
Uniform Policy and Method for Soils Evaluation, Testing, and Reporting , Percolation Testing.

Time	T_1 (min.)	H_1 (in.)	H_2 (in.)	D (in.)	t (min./in.)
11:45	60	33.000	33.000	0.000	0.00
12:45					
12:45	60	33.000	33.125	0.125	480.00
13:45					
13:45	60	33.125	33.125	0.000	0.00
14:45					
14:45	60	33.125	33.250	0.125	480.00
15:45					
15:45	60	33.250	33.375	0.125	480.00
16:45					

Stabilized rate of drop, t= 480.00

Reported Percolation Rate, P_u= 0.23 gal/ft²/day

T_1	Time Interval
H_1	Initial Water Level
H_2	Final Water Level
D	Change In Water Level
t	Rate of Drop (min/in)

$$P_u = 5 / (t)^{1/2}$$

P_u= Percolation rate (gal/ft²/day)
t= Stabilized rate of drop (min/in)

Geotechnics Incorporated

PERCOLATION TEST RESULTS

Project No. 0554-075-00
Document No. 06-0015
FIGURE C-1

Test Hole No. PT-2 Date Excavated 1/17/2006
Depth of Test Hole 5 feet Earth Material Clay/Fat clay CL/CH
Actual Percolation Tested by JSO Date 1/20/2006 Test Hole Diameter 8 in.
Case Number

Reference: Imperial County Public Health Department, Division of Environmental Health Services, *Uniform Policy and Method for Soils Evaluation, Testing, and Reporting* , Percolation Testing.

Time	T_1 (min.)	H_1 (in.)	H_2 (in.)	D (in.)	t (min./in.)
11:27 12:27	60	21.188	21.188	0.000	0.00
12:27 13:27	60	21.188	21.188	0.000	0.00
13:27 14:27	60	21.188	21.188	0.000	0.00
14:27 15:27	60	21.188	21.188	0.000	0.00
15:27 16:27	60	21.188	21.188	0.000	0.00

Stabilized rate of drop, t= 0.00

Reported Percolation Rate, P_u= 0.00 gal/ft²/day

T_1	Time Interval
H_1	Initial Water Level
H_2	Final Water Level
D	Change In Water Level
t	Rate of Drop (min/in)

$$P_u = 5 / (t)^{1/2}$$

P_u= Percolation rate (gal/ft²/day)
t= Stabilized rate of drop (min/in)

Geotechnics Incorporated

PERCOLATION TEST RESULTS

FIGURE C-2

Test Hole No.	PT-3	Date Excavated	1/17/2006		
Depth of Test Hole	5 feet	Earth Material	Clay/Fat clay CL/CH		
Actual Percolation Tested by	JSO	Date	1/20/2006	Test Hole Diameter	8 in.
Case Number					

Reference: Imperial County Public Health Department, Division of Environmental Health Services, *Uniform Policy and Method for Soils Evaluation, Testing, and Reporting* , Percolation Testing.

Time	T_1 (min.)	H_1 (in.)	H_2 (in.)	D (in.)	t (min./in.)
11:00 12:00	60	20.188	20.313	0.125	0.00
12:00 13:00	60	20.313	20.375	0.063	960.00
13:00 14:00	60	20.375	20.375	0.000	0.00
14:00 15:00	60	20.375	20.438	0.063	960.00
15:00 16:00	60	20.438	20.500	0.063	960.00

Stabilized rate of drop, t= 960.00

Reported Percolation Rate, P_U= 0.16 gal/ft²/day

T_1 Time Interval

H_1 Initial Water Level

H_2 Final Water Level

D Change In Water Level

t Rate of Drop (min/in)

$$P_U = 5 / (t)^{1/2}$$

P_U= Percolation rate (gal/ft²/day)

t= Stabilized rate of drop (min/in)

APPENDIX D

FIELD RESISTIVITY TESTING

The results of the *soil* and *thermal* resistivity testing conducted at the site are presented in Tables 1 through 3 of Appendix D. The approximate locations of the boreholes corresponding to the field resistivity tests (BH-9, BH-11, BH-12, BH-14 and BH-17) are shown on the Exploration Plan, Figure 2. All resistivity tests were conducted by Schiff Associates between January 27 and 30, 2006. Please contact Mr. James T. Keegan with Schiff Associates with any questions or comments regarding the test results presented in this appendix.

The in-situ *soil* resistivity testing was conducted at three locations within each of the areas for the two turbine generators (BH-11 and BH-14), GSU (BH-9) and switchyard (BH-17) for a total of 12 locations. The *soil* resistivity tests were conducted at depths of 2½, 5, 7½, 10 and 15 feet, and are summarized in Table 2 of Appendix D. It is our understanding that the *soil* resistivity testing was conducted in general accordance with IEEE Standard 81. The in-situ *thermal* resistivity testing was conducted at two locations (BH-12 and BH-17). The *thermal* resistivity tests are summarized in Table 3 of Appendix D. It is our understanding that the *thermal* resistivity tests were conducted in general accordance with IEEE Standard 442.

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Table 1 - Laboratory Tests on Soil Samples

Geotechnics, Inc.
Imperial Irrigation District Gas Turbine Plant, Niland, CA
MJS&A #06-0146ENG
30-Jan-06

Sample ID			BH-17 @ 0-5'	BH-12 @ 0-5'	BH-1C @ 0-3'	BH-6C @ 0-5'	BH-5C @ 0-5'
Resistivity		**Units**					
as-received		ohm-cm	800,000	260,000	71,000	210,000	170,000
saturated		ohm-cm	510	640	320	200	400
pH			7.7	7.6	7.9	7.7	7.8
Electrical Conductivity		mS/cm	0.79	0.68	1.47	2.00	0.75
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	297	164	405	878	240
magnesium	Mg^{2+}	mg/kg	141	78	124	75	148
sodium	Na^{1+}	mg/kg	184	409	968	873	222
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	229	265	198	168	232
chloride	Cl^{1-}	mg/kg	460	390	1,090	980	530
sulfate	SO_4^{2-}	mg/kg	850	818	1,849	2,765	726
Other Tests							
ammonium	NH_4^{1+}	mg/kg	5.4	1.3	2.1	2.1	1.5
nitrate	NO_3^{1-}	mg/kg	133.2	3.5	12.4	62.0	31.2
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

Table 1 - Laboratory Tests on Soil Samples

Geotechnics, Inc.
Imperial Irrigation District Gas Turbine Plant, Niland, CA
MJS&A #06-0146ENG
30-Jan-06

Sample ID			BH-3C @ 0-5'	BH-9C @ 0-5'
Resistivity		**Units**		
as-received		ohm-cm	62,000	150,000
saturated		ohm-cm	250	220
pH			8.0	7.8
Electrical Conductivity		mS/cm	1.97	1.34
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	561	493
magnesium	Mg^{2+}	mg/kg	168	153
sodium	Na^{1+}	mg/kg	1,115	600
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	272	241
chloride	Cl^{1-}	mg/kg	890	640
sulfate	SO_4^{2-}	mg/kg	2,917	1,984
Other Tests				
ammonium	NH_4^{1+}	mg/kg	2.6	2.7
nitrate	NO_3^{1-}	mg/kg	ND	33.4
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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Test Date 01-27-06

TABLE - 2
SOIL RESISTIVITY - FIELD TESTS
GEOTECHNICS - TURBINE GENERATOR, NILAND, CA
MJS&A# 06-0146ENG

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)	DEPTH TO PIPE CENTERLINE (feet)
Turbine Gen.1				100000	
BH10-11-a	2.5	200.00	100000		
				29032	
	5.0	45.00	45000		
				11250	
	7.5	15.00	22500		
				30000	
	10.0	12.00	24000		
				24000	
	15.0	8.00	24000		
Turbine Gen.1				800000	
BH10-11-b	2.5	1600.00	800000		
				284746	
	5.0	420.00	420000		
				13291	
	7.5	25.00	37500		
	10.0	NR			
	15.0	NR			
Turbine Gen.1				900000	
BH10-11-c	2.5	1800.00	900000		
				572727	
	5.0	700.00	700000		
				13501	
	7.5	26.00	39000		
	10.0	NR			
	15.0	NR			
GSU				85000	
BH9-a	2.5	170.00	85000		
				110270	
	5.0	96.00	96000		
	7.5	NR			
	10.0	NR			
	15.0	NR			

NR = Not Read due to interference or meter sensitvity.

Test Date 01-27-06

TABLE - 2
SOIL RESISTIVITY - FIELD TESTS
GEOTECHNICS - TURBINE GENERATOR, NILAND, CA
MJS&A# 06-0146ENG

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)	DEPTH TO PIPE CENTERLINE (feet)
GSU				550000	
BH9-b	2.5	1100.00	550000		
				2811111	
	5.0	920.00	920000		
				23657	
	7.5	45.00	67500		
				19688	
	10.0	21.00	42000		
	15.0	NR			
GSU				1550000	
BH9-c	2.5	3100.00	1550000		
				852500	
	5.0	1100.00	1100000		
				15949	
	7.5	31.00	46500		
	10.0	NR			
	15.0	NR			
Switch Yard				55000	
BH17-18-a	2.5	110.00	55000		
				6735	
	5.0	12.00	12000		
	7.5	NR			
	10.0	NR			
	15.0	NR			
Switch Yard				26000	
BH17-18-b	2.5	52.00	26000		
				28080	
	5.0	27.00	27000		
				19636	
	7.5	16.00	24000		
	10.0	NR			
	15.0	NR			

NR = Not Read due to interference or meter sensitvity.

Test Date 01-27-06

TABLE - 2
SOIL RESISTIVITY - FIELD TESTS
GEOTECHNICS - TURBINE GENERATOR, NILAND, CA
MJS&A# 06-0146ENG

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)	DEPTH TO PIPE CENTERLINE (feet)
Switch Yard				27500	
BH17-18-c	2.5	55.00	27500		
				4888	
	5.0	8.30	8300		
				3860	
	7.5	4.00	6000		
	10.0	NR			
	15.0	NR			
Turbine Gen.2				7500	
BH14-15-a	2.5	15.00	7500		
				1154	
	5.0	2.00	2000		
				5667	
	7.5	1.70	2550		
	10.0	NR			
	15.0	NR			
Turbine Gen.2				1000000	
BH14-15-b	2.5	2000.00	1000000		
				162791	
	5.0	280.00	280000		
				7368	
	7.5	14.00	21000		
				597	
	10.0	1.10	2200		
	15.0	NR			
Turbine Gen.2				110000	
BH14-15-c	2.5	220.00	110000		
				6908	
	5.0	13.00	13000		
				6913	
	7.5	6.70	10050		
	10.0	NR			
	15.0	NR			

NR = Not Read due to interference or meter sensitvity.

Table 3 - Field Thermal Resistivity Results

Geotechnics, Inc.
Imperial Irrigation District Gas Turbine Plant, Niland, CA
MJS&A #06-0146ENG
30-Jan-06

Sample ID			
		BH-12	BH-17
Thermal Resistivity	**Units**		
	M-°C/W	2.14	3.70

APPENDIX E

LABORATORY TESTING

Laboratory testing was conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions and in the same locality. No warranty, express or implied, is made as to the correctness or serviceability of the test results, or the conclusions derived from these tests. Where a specific laboratory test method has been referenced, such as ASTM, Caltrans, or AASHTO, the reference applies only to the specified laboratory test method and not to associated referenced test method(s) or practices, and the test method referenced has been used only as a guidance document for the general performance of the test and not as a "Test Standard". A brief description of the tests performed follows.

Classification: Soils were classified visually according to the Unified Soil Classification System as established by the American Society of Civil Engineers. Visual classification was supplemented by laboratory testing of selected soil samples and classification in general accordance with the laboratory soil classification tests outlined in ASTM test method D2487. The resultant soil classifications are shown on the boring logs in Appendix B.

Particle Size Analysis: Particle size analyses were performed in general accordance with ASTM D422, and were used to supplement visual soil classifications. The results are presented in Figures E-1.1 through E-1.18.

Atterberg Limits: ASTM D4318 was used to determine the liquid and plastic limits, and plasticity index of selected soils. The results are also shown in selected Figures E-1.1 through E-1.18.

In-Situ Moisture/Density: The in-place moisture contents and dry unit weights of selected soil samples were determined using relatively undisturbed samples from the liner rings of the Modified California sampler. The dry unit weights and moisture contents are shown on the boring logs.

Maximum Density/Optimum Moisture: The maximum dry densities and optimum moisture contents of selected soil samples were determined using ASTM D1557 as a guideline. The test results are summarized in Figure E-2.

Expansion Index: The expansion potential of selected soils was estimated in general accordance with the laboratory procedures outlined in ASTM test method D4829. The test results are summarized on Figure E-3. Figure E-3 also presents the UBC criteria for evaluating the expansion potential based on the expansion index.

APPENDIX E

LABORATORY TESTING (Continued)

Sulfate Content: To assess the potential for reactivity with concrete, soil samples were tested for water soluble sulfate. The sulfate was extracted from the soil under vacuum, typically using a 20:1 (water to dry soil) dilution ratio. The extracted solution was tested for water soluble sulfate in general accordance with ASTM D516. The test results are presented in Figure E-4. Figure E-4 also presents the UBC criteria for evaluating soluble sulfate content.

Chloride Content: Soil samples were also tested for water soluble chloride. The chloride was extracted from the soil under vacuum, typically using a 20:1 (water to dry soil) dilution ratio. The extracted solution was then tested for water soluble chloride using a calibrated ion specific electronic probe (Orion 710A+). The test results are also shown in Figure E-4.

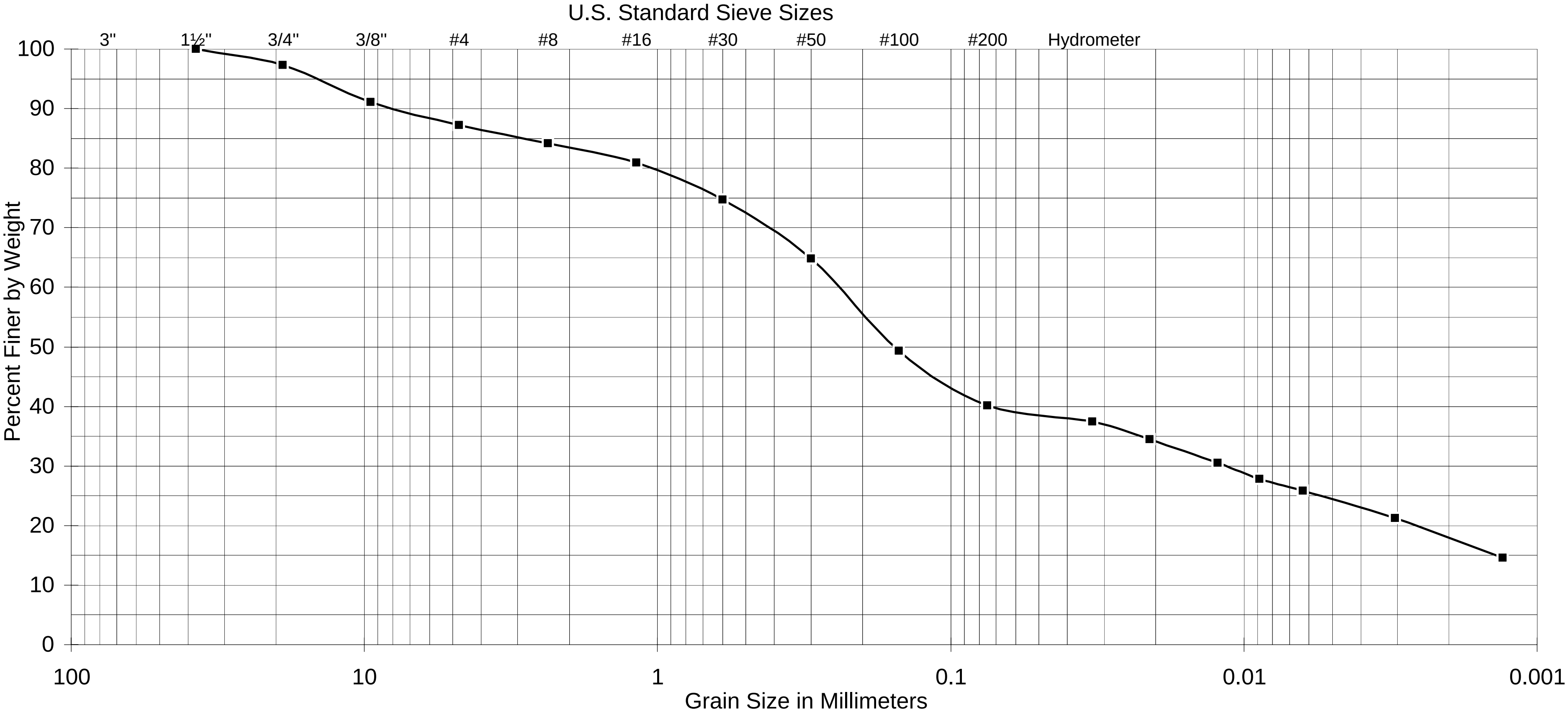
pH and Resistivity: To assess the potential for reactivity with metal, representative samples were tested for pH and resistivity using Caltrans method 643. The results are also given in Figure E-4.

Direct Shear: The shear strength of selected soil samples was assessed using direct shear testing performed in general accordance with ASTM D3080. The direct shear test results are shown in Figures E-5.1 through E-5.3.

Unconfined Compression: The undrained compressive strength of selected soil samples was assessed using unconfined compression testing performed in general accordance with ASTM D32166. The test results are shown in Figures E-6.1 and E-6.2.

Consolidation Test: In order to aid in evaluating soil compressibility, one-dimensional consolidation tests were conducted in general accordance with the laboratory procedures outlined in ASTM test method D2435. The soil samples were restrained laterally and drained axially. The first two unsaturated soil samples were inundated with water at a nominal seating load, allowed to swell, and then subjected to incremental controlled stress loading. The second two samples were loaded at the in-situ moisture content (no water was added). The results are shown in Figures E-7.1 to E-7.4.

R-Value: To aid in developing preliminary pavement section designs, two R-Value tests were performed on selected soil samples in general accordance with California Test Method 301. Both tests indicated an R-Value of 5 or less. The R-Value test results are discussed in Section 9.8.



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-1
SAMPLE LOCATION:	0' - 3'

UNIFIED SOIL CLASSIFICATION: SC

DESCRIPTION: CLAYEY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	32
PLASTIC LIMIT:	13
PLASTICITY INDEX:	19

Geotechnics Incorporated

SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.1

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-1
SAMPLE LOCATION:	5' - 6½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	
PLASTIC LIMIT:	
PLASTICITY INDEX:	

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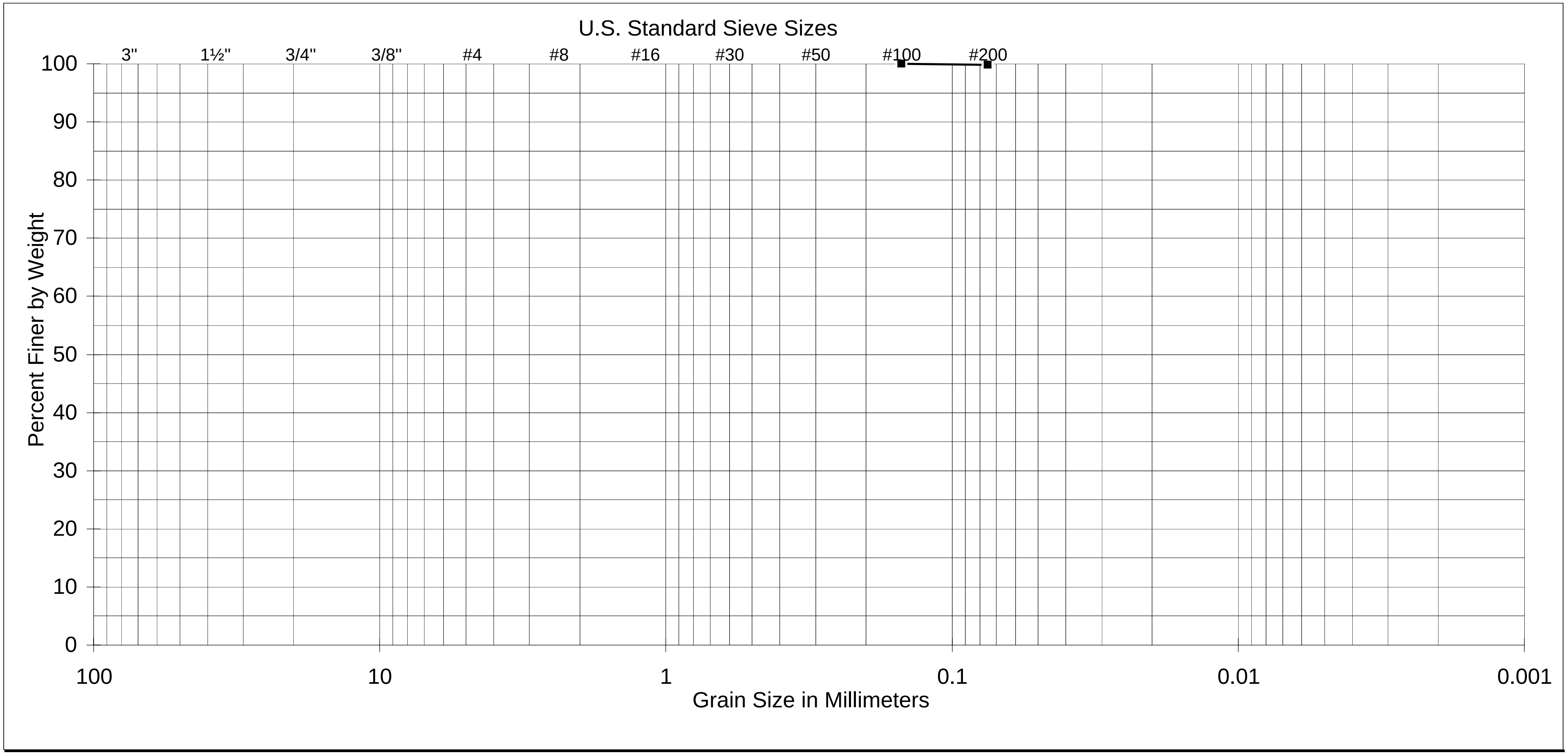
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

SAMPLE NUMBER: BH-1

SAMPLE LOCATION: 15' - 16½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS

LIQUID LIMIT:

PLASTIC LIMIT:

PLASTICITY INDEX:

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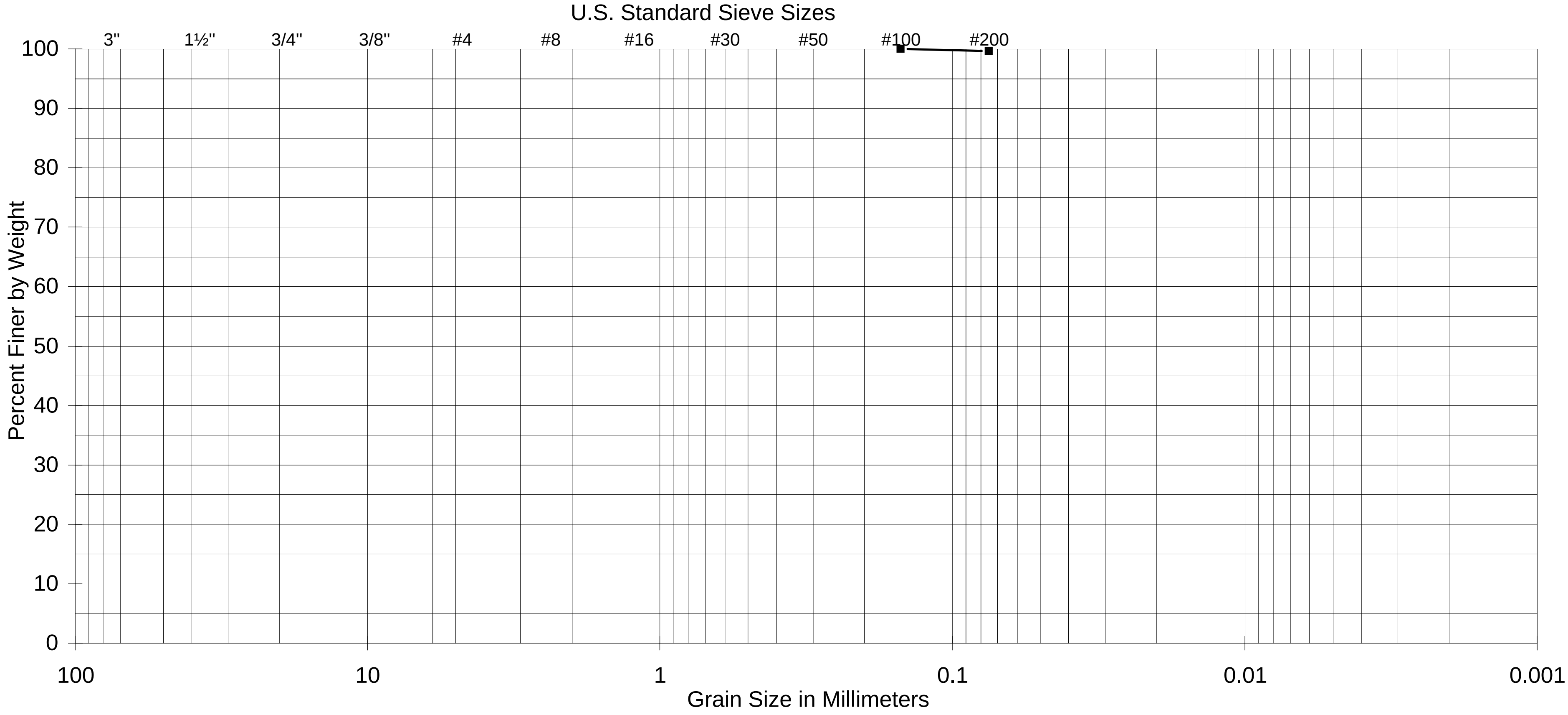
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.3



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	BH-1
SAMPLE LOCATION:	25' - 26½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	
PLASTIC LIMIT:	
PLASTICITY INDEX:	

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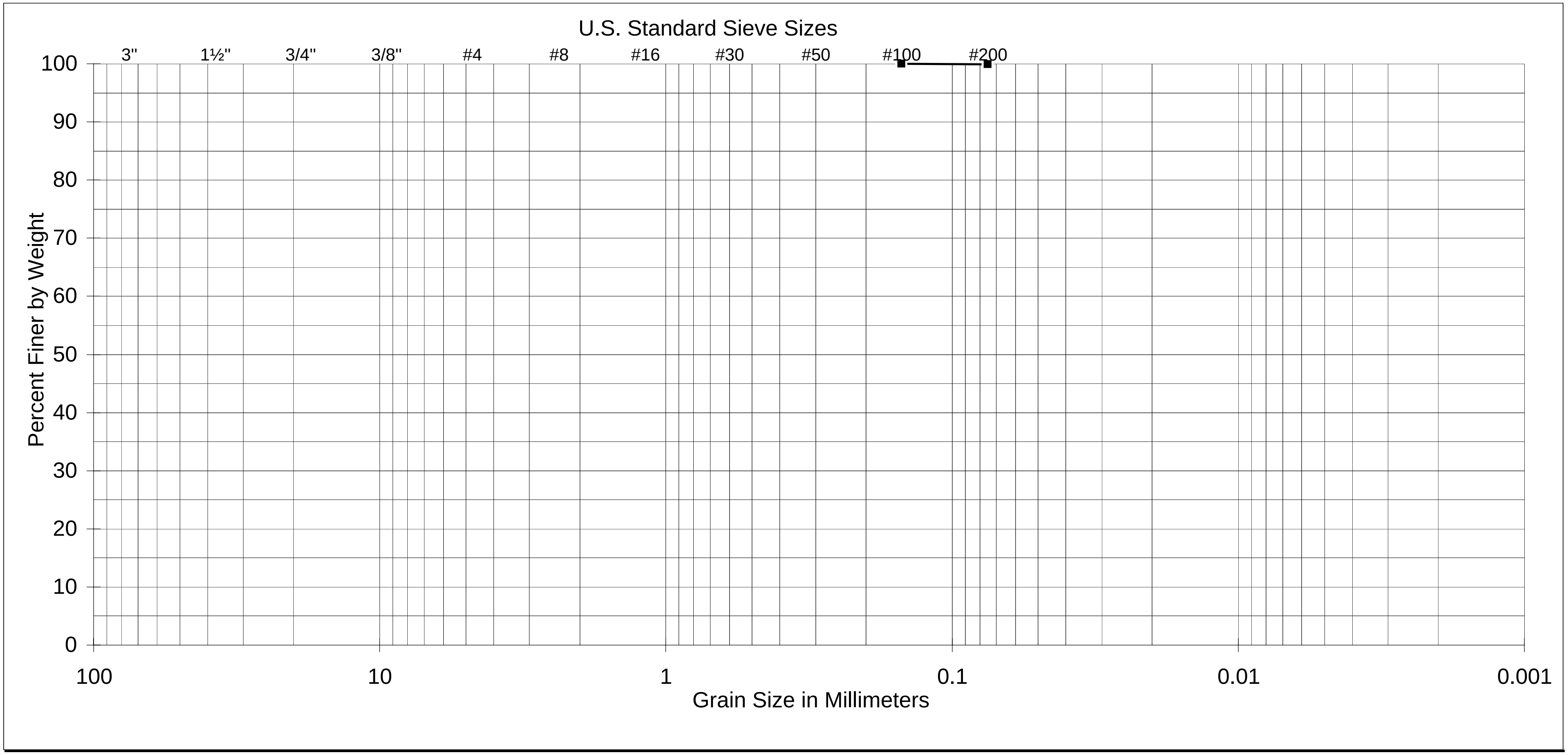
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.4



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

SAMPLE NUMBER: BH-1

SAMPLE LOCATION: 35' - 36½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS

LIQUID LIMIT:

PLASTIC LIMIT:

PLASTICITY INDEX:

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SOIL CLASSIFICATION

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FIGURE E-1.5

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	BH-1
SAMPLE LOCATION:	45' - 46½'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	
PLASTIC LIMIT:	
PLASTICITY INDEX:	

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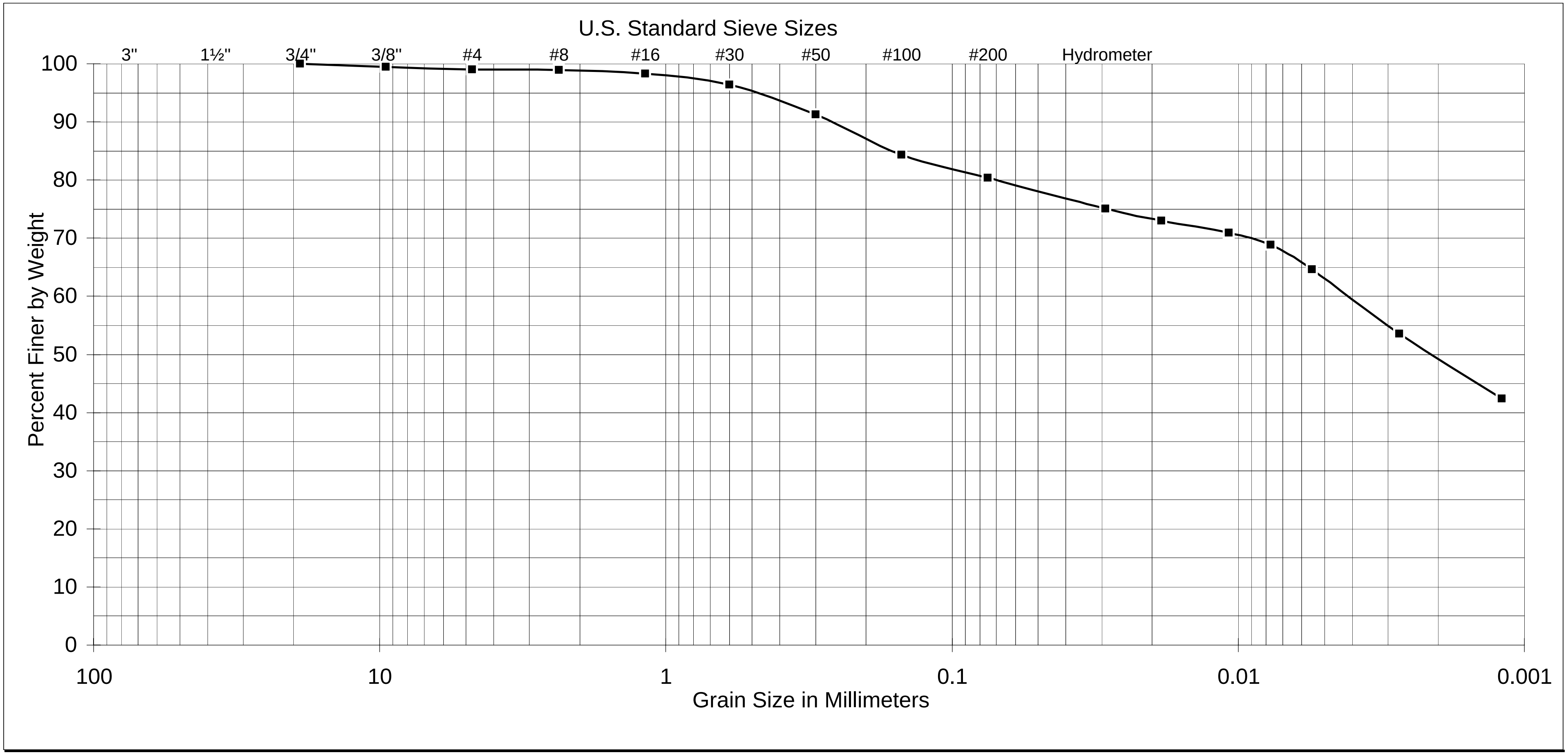
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.6



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-3
SAMPLE LOCATION:	0' - 5'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY WITH SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	61
PLASTIC LIMIT:	21
PLASTICITY INDEX:	40

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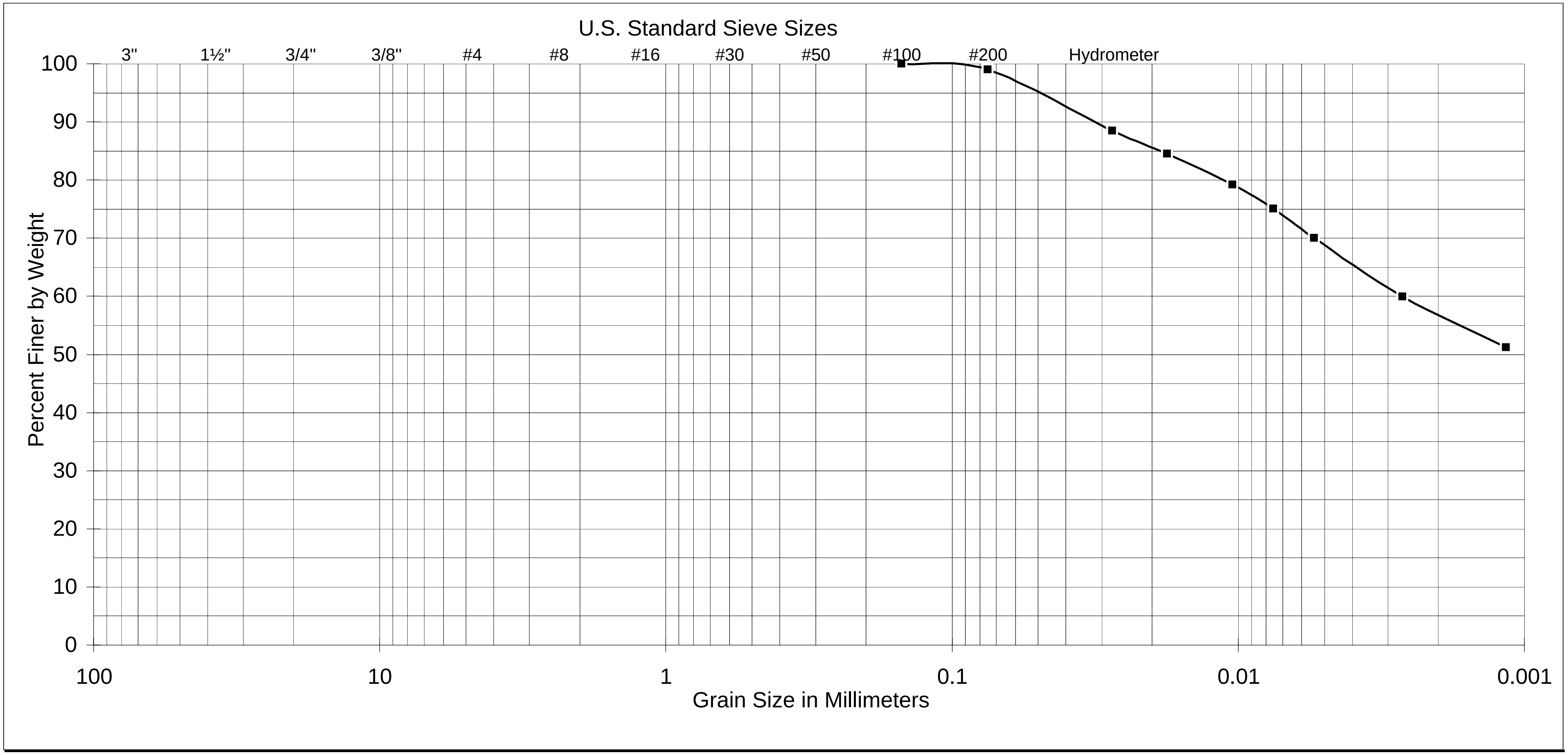
Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.7



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-5
SAMPLE LOCATION:	8' - 10'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	71
PLASTIC LIMIT:	21
PLASTICITY INDEX:	50

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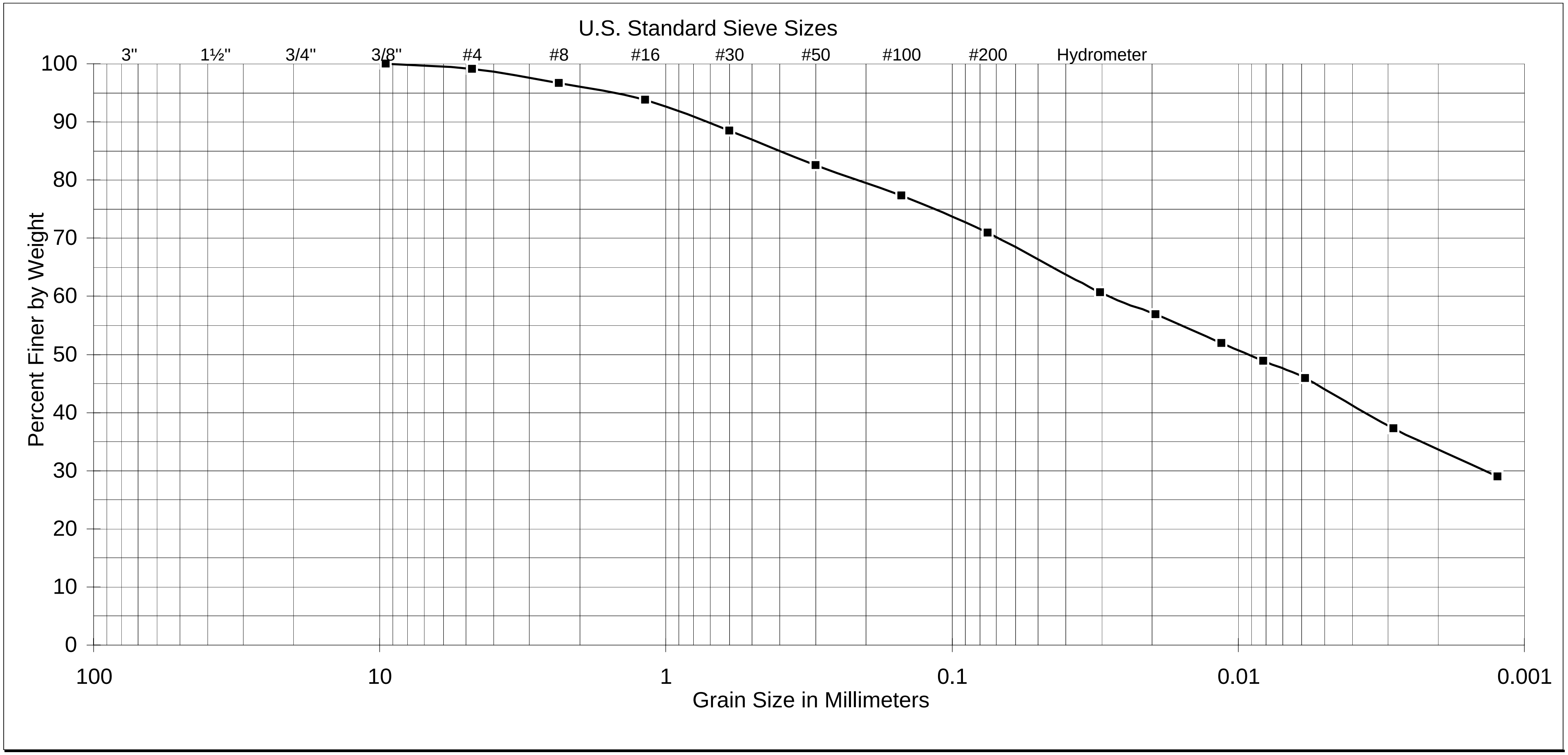
FIGURE E-1.8



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-6
SAMPLE LOCATION:	0' - 5'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: SANDY LEAN CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	37
PLASTIC LIMIT:	15
PLASTICITY INDEX:	22

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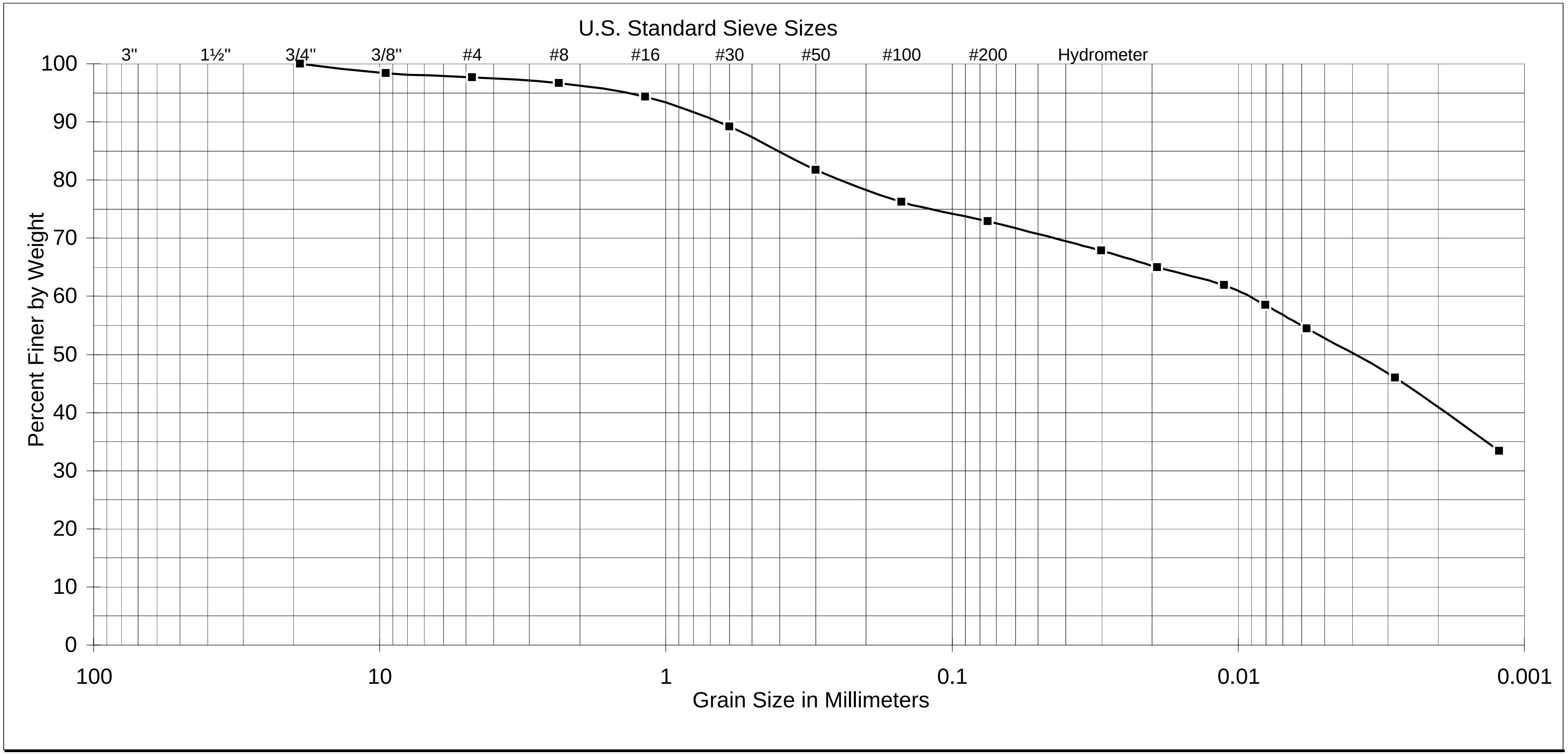
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.9



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

SAMPLE NUMBER: BH-9

SAMPLE LOCATION: 0' - 5'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY WITH SAND

ATTERBERG LIMITS

LIQUID LIMIT: 55

PLASTIC LIMIT: 20

PLASTICITY INDEX: 35

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FIGURE E-1.10

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
SAMPLE NUMBER:	BH-12
SAMPLE LOCATION:	0' - 5'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: SANDY CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	31
PLASTIC LIMIT:	15
PLASTICITY INDEX:	16

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Document No. 06-0015
FIGURE E-1.11

COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

SAMPLE NUMBER: BH-12

SAMPLE LOCATION: 5' - 6'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY WITH SAND

ATTERBERG LIMITS

LIQUID LIMIT:

PLASTIC LIMIT:

PLASTICITY INDEX:

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FIGURE E-1.12

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE

SAMPLE NUMBER: BH-12

SAMPLE LOCATION: 15' - 16'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS

LIQUID LIMIT:

PLASTIC LIMIT:

PLASTICITY INDEX:

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SOIL CLASSIFICATION

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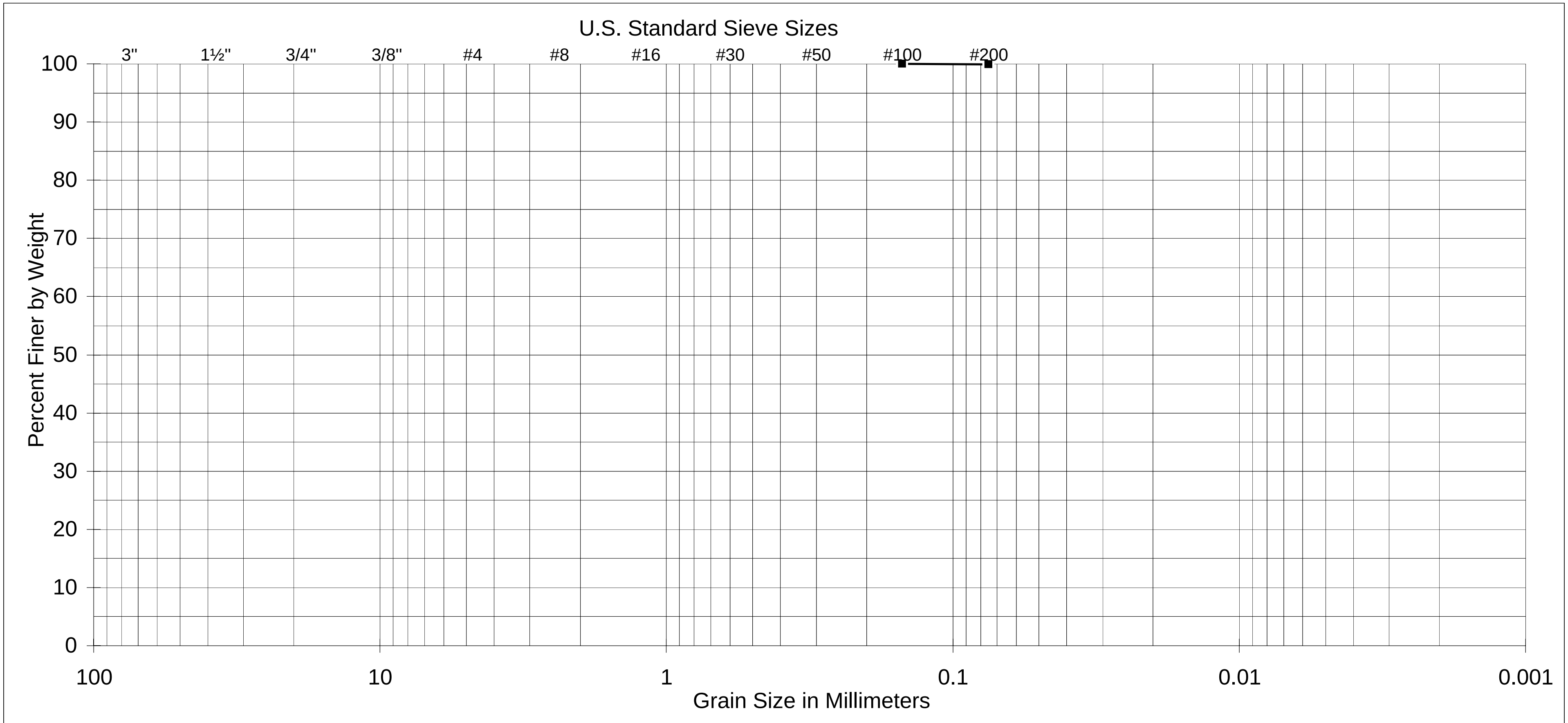
FIGURE E-1.13



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

SAMPLE NUMBER: BH-12

SAMPLE LOCATION: 25' - 26'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS

LIQUID LIMIT:

PLASTIC LIMIT:

PLASTICITY INDEX:

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SOIL CLASSIFICATION

Project No. 0554-075-00
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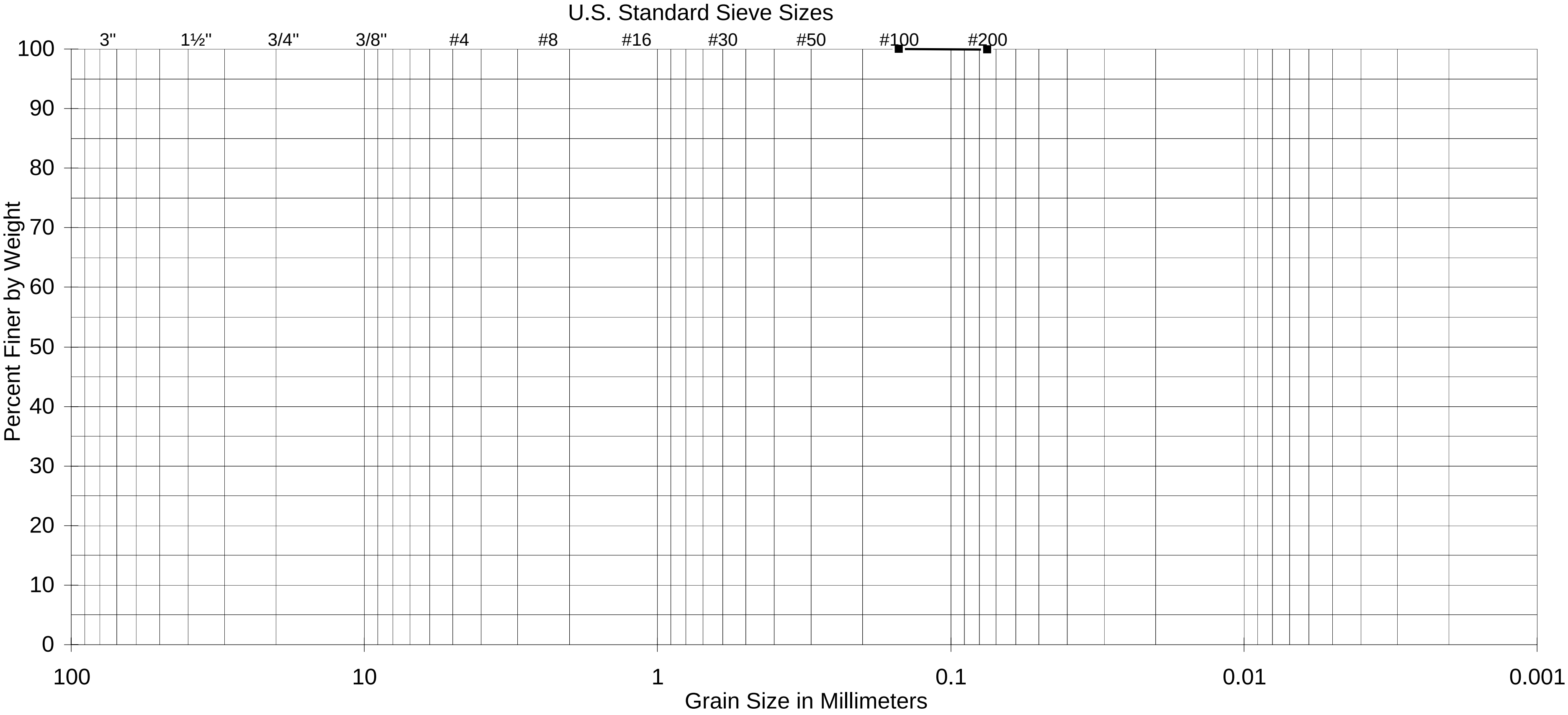
FIGURE E-1.14



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-13
SAMPLE LOCATION:	10' - 11'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

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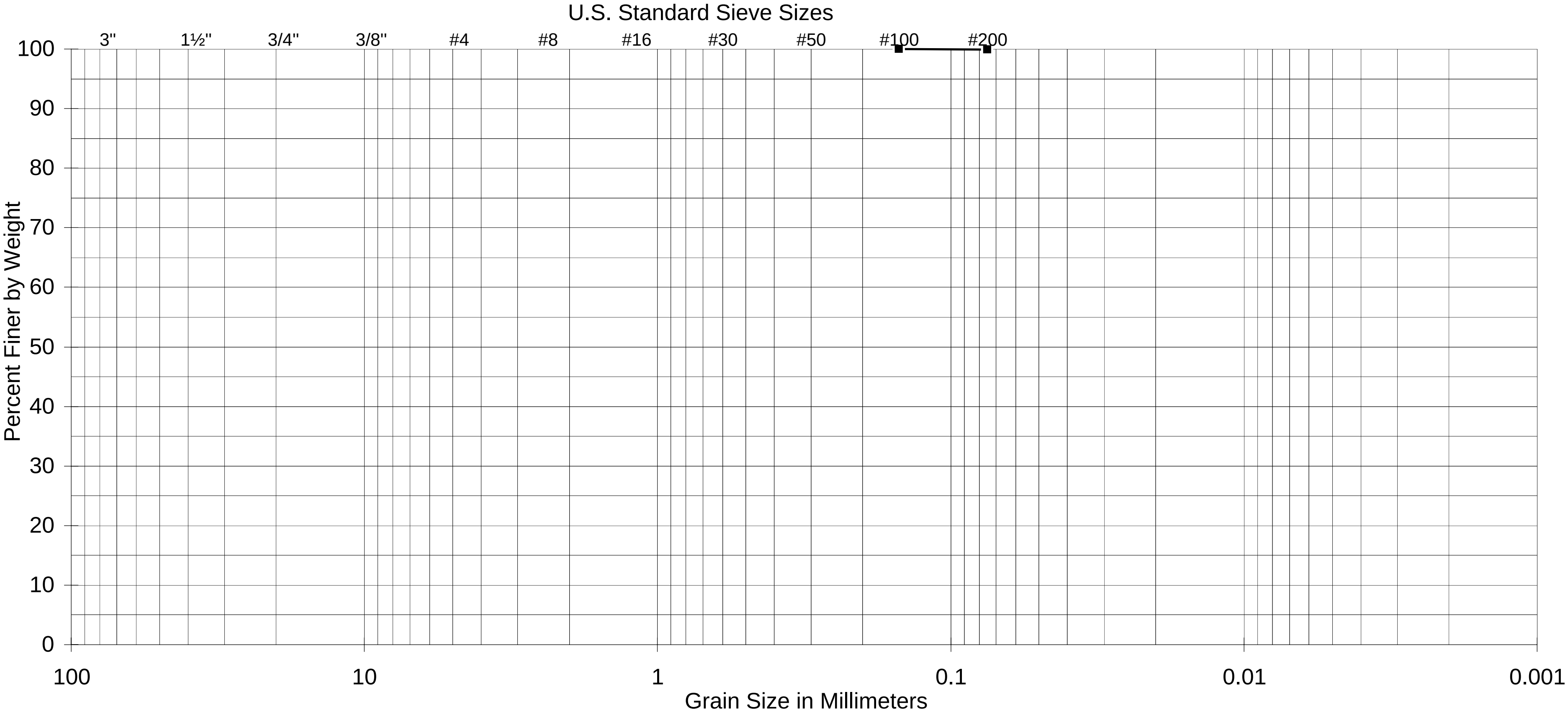
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.15



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-13
SAMPLE LOCATION:	20' - 21'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

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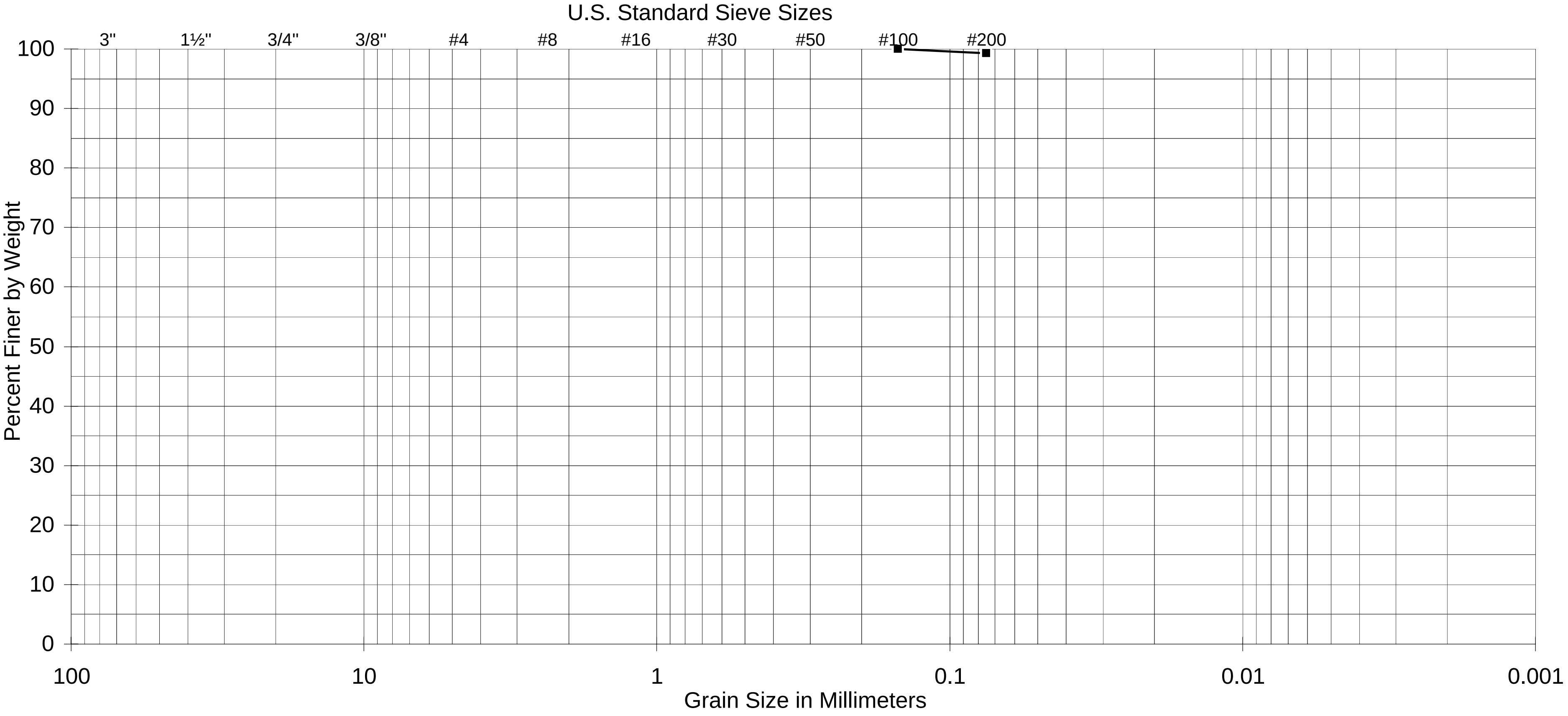
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.16



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	BH-13
SAMPLE LOCATION:	30' - 31'

UNIFIED SOIL CLASSIFICATION: CH

DESCRIPTION: FAT CLAY

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

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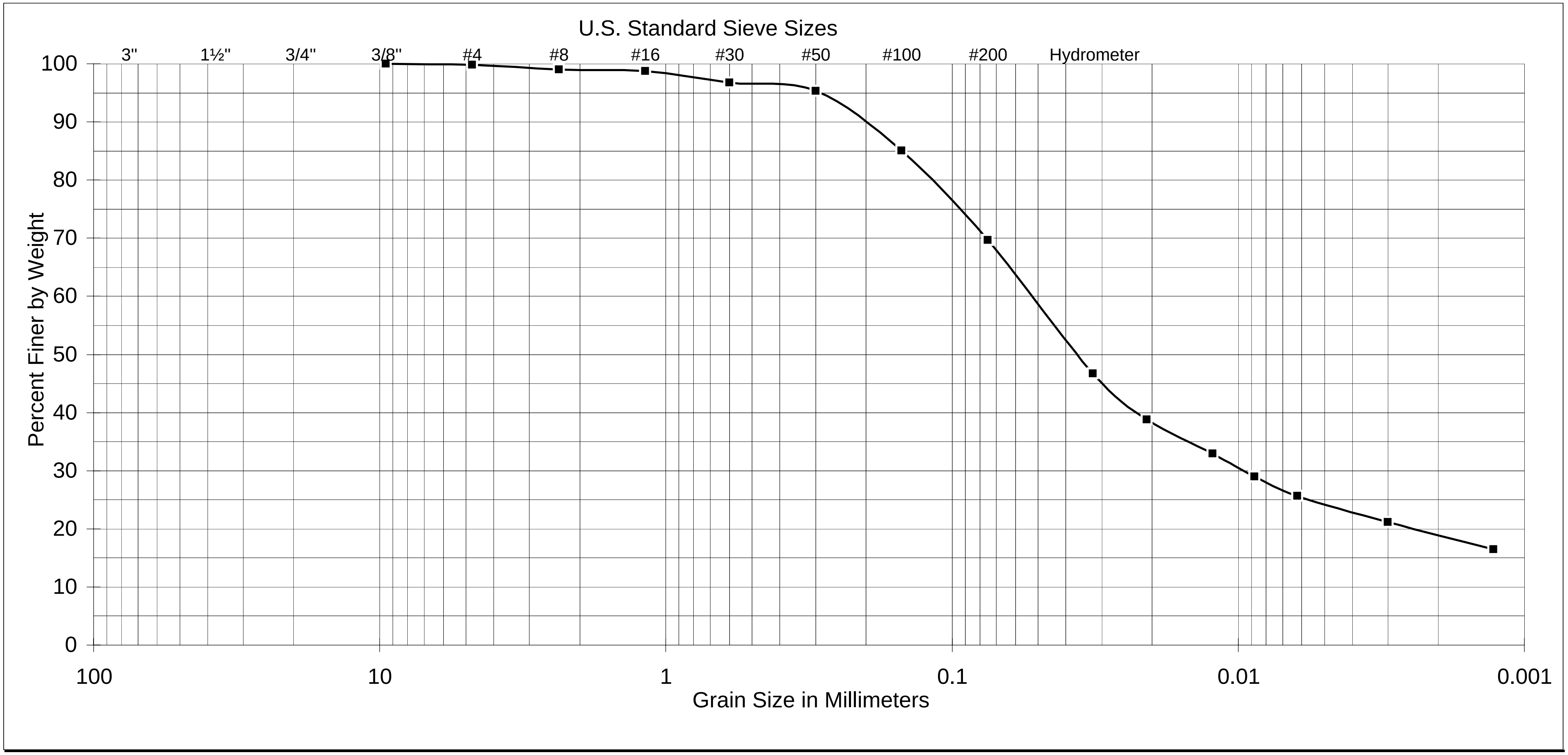
SOIL CLASSIFICATION

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-1.17



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE

SAMPLE NUMBER: BH-17

SAMPLE LOCATION: 0' - 5'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: SANDY LEAN CLAY

ATTERBERG LIMITS

LIQUID LIMIT: 27

PLASTIC LIMIT: 16

PLASTICITY INDEX: 11

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SOIL CLASSIFICATION

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Document No. 06-0015

FIGURE E-1.18

MAXIMUM DENSITY TEST RESULTS

(ASTM D1557)

SAMPLE	DESCRIPTION	MAXIMUM DENSITY [PCF]	OPTIMUM MOISTURE [%]
BH-3 @ 0’ – 5’	Brown fat clay with sand (CH).	114½	16½
BH-9 @ 0’ – 5’	Brown fat clay with sand (CH).	116	15½

EXPANSION TEST RESULTS

(ASTM D4829)

SAMPLE	DESCRIPTION	EXPANSION INDEX
BH-1 @ 0’ – 3’	Yellow brown clayey sand (SC).	69
BH-3 @ 0’ – 5’	Brown fat clay with sand (CH).	109
BH-5 @ 8’ – 10’	Dark brown fat clay (CH).	106
BH-6 @ 0’ – 5’	Yellow brown sandy lean clay (CL).	85
BH-9 @ 0’ – 5’	Brown fat clay with sand (CH).	97
BH-12 @ 0’ – 5’	Dark yellow brown sandy lean clay (CL).	43
BH-17 @ 0’ – 5’	Dark yellow brown sandy lean clay (CL).	29

UBC TABLE NO. 18-1-B, CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very High

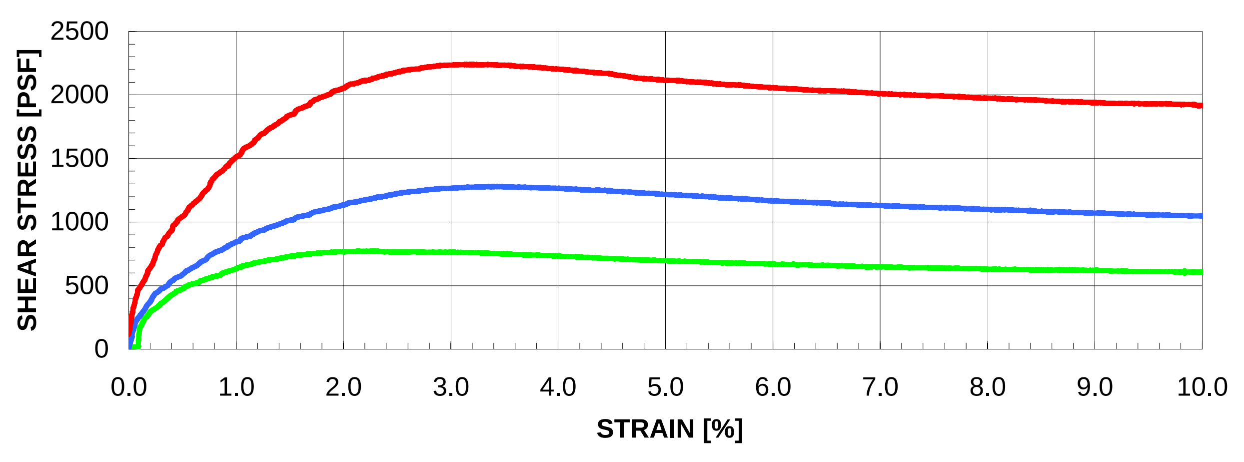
CHEMISTRY TEST RESULTS
(ASTM D516, CTM 643)

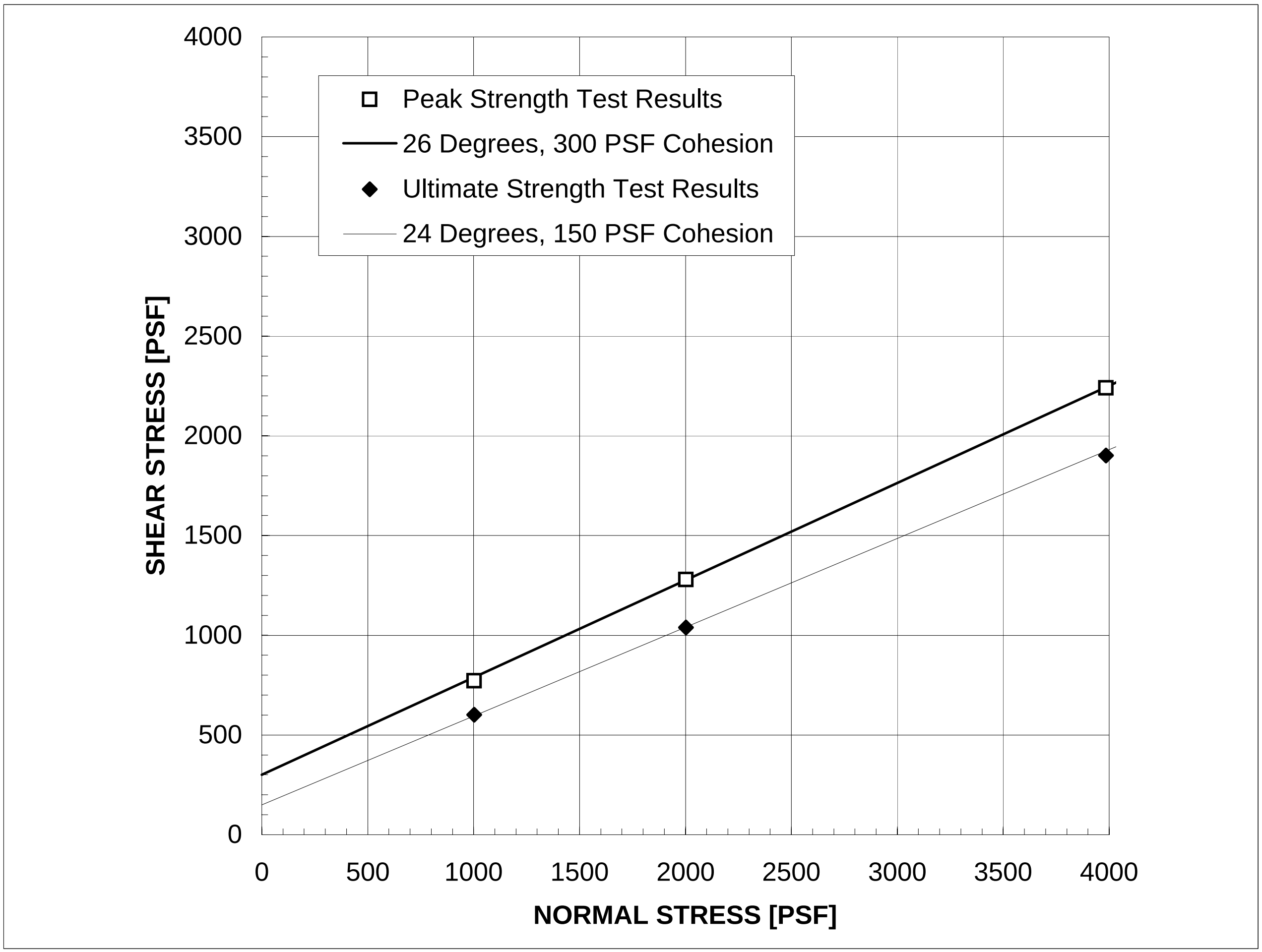
SAMPLE	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
BH-1 @ 0’ – 3’	8.1	190	0.49	0.16
BH-6 @ 0’ – 5’	7.9	230	0.74	0.10
BH-12 @ 0’ – 5’	8.1	400	0.11	0.04
BH-17 @ 0’ – 5’	7.9	380	0.14	0.05

SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

SOIL RESISTIVITY [OHM-CM]	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT [%]	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive





SAMPLE: BH-12 @ 5' - 6'

LACUSTRINE DEPOSITS:

Dark brown fat clay with sand (CH).

	PEAK	**ULTIMATE**
ϕ'	26 °	24 °
C'	300 PSF	150 PSF
	IN-SITU	**AS-TESTED**
γ_d	106.3 PCF	106.3 PCF
w_c	13.7 %	22.4 %

STRAIN RATE: 0.0002 IN/MIN

(Sample was consolidated and drained)

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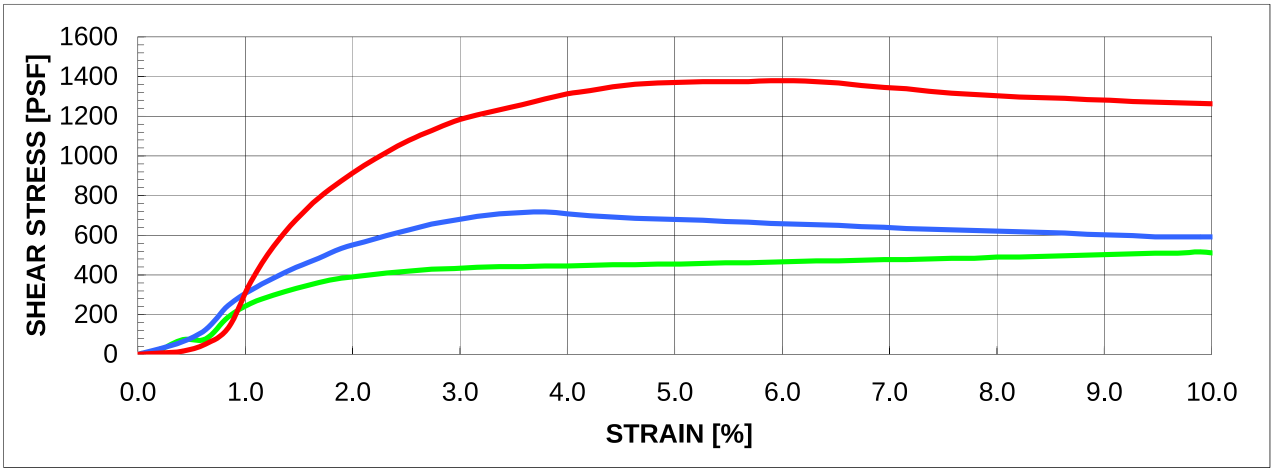
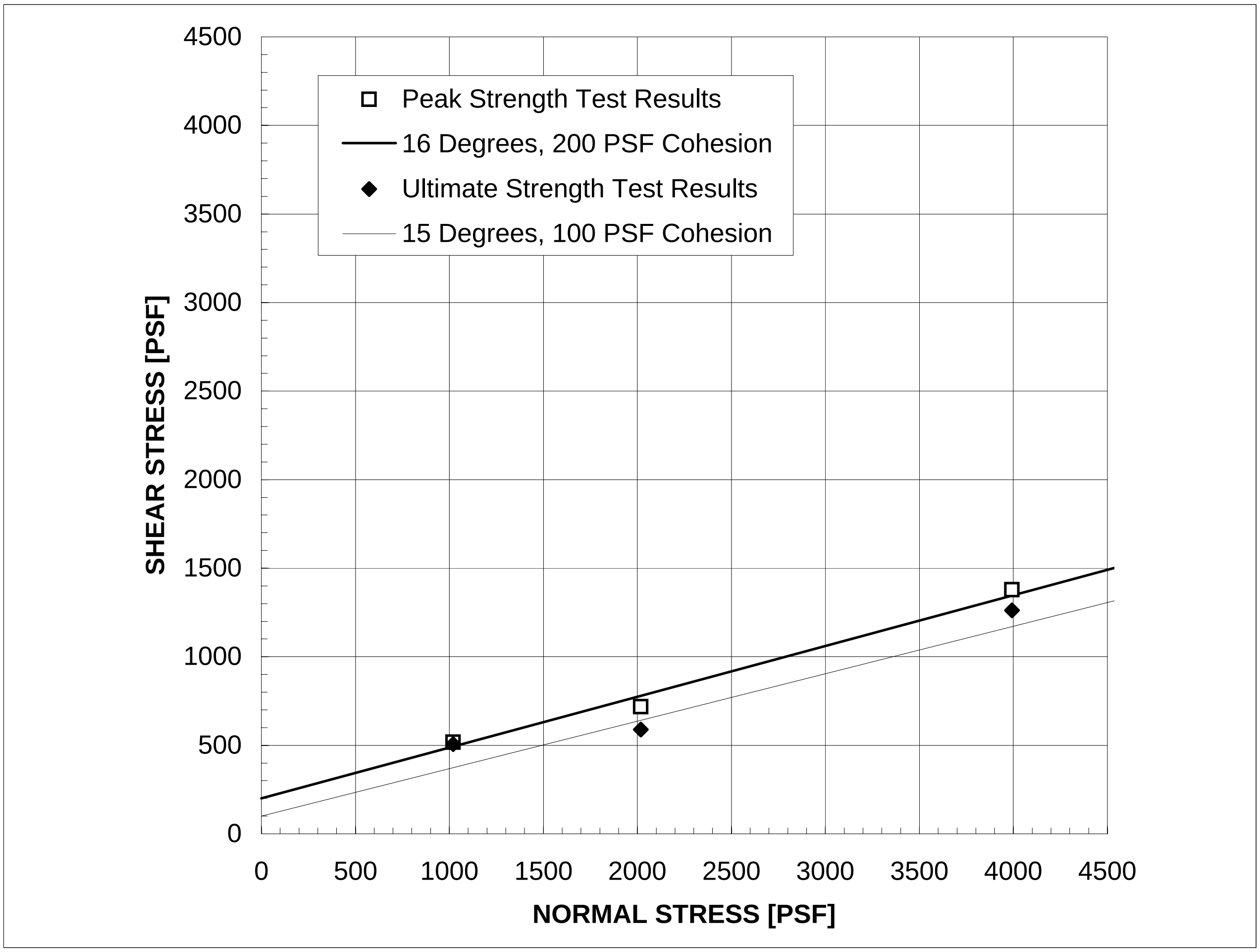
DIRECT SHEAR TEST RESULTS

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-5.1





SAMPLE: BH-12 @ 15' - 16'

LACUSTRINE DEPOSITS:
Dark brown fat clay (CH).

	PEAK	ULTIMATE
ϕ'	16 °	15 °
C'	200 PSF	100 PSF

STRAIN RATE: 0.0002 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	105.9 PCF	105.9 PCF
w_c	18.9 %	23.2 %

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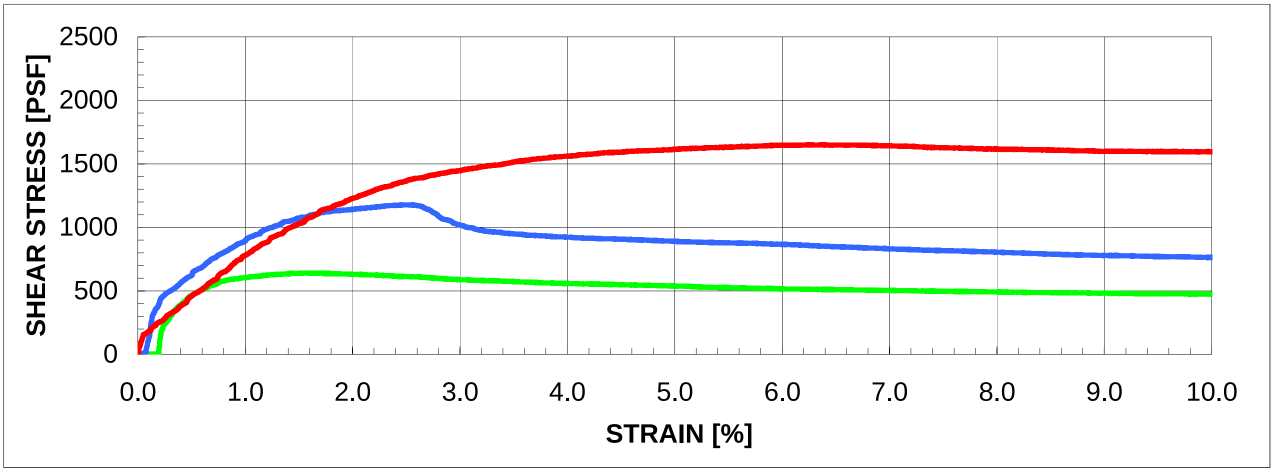
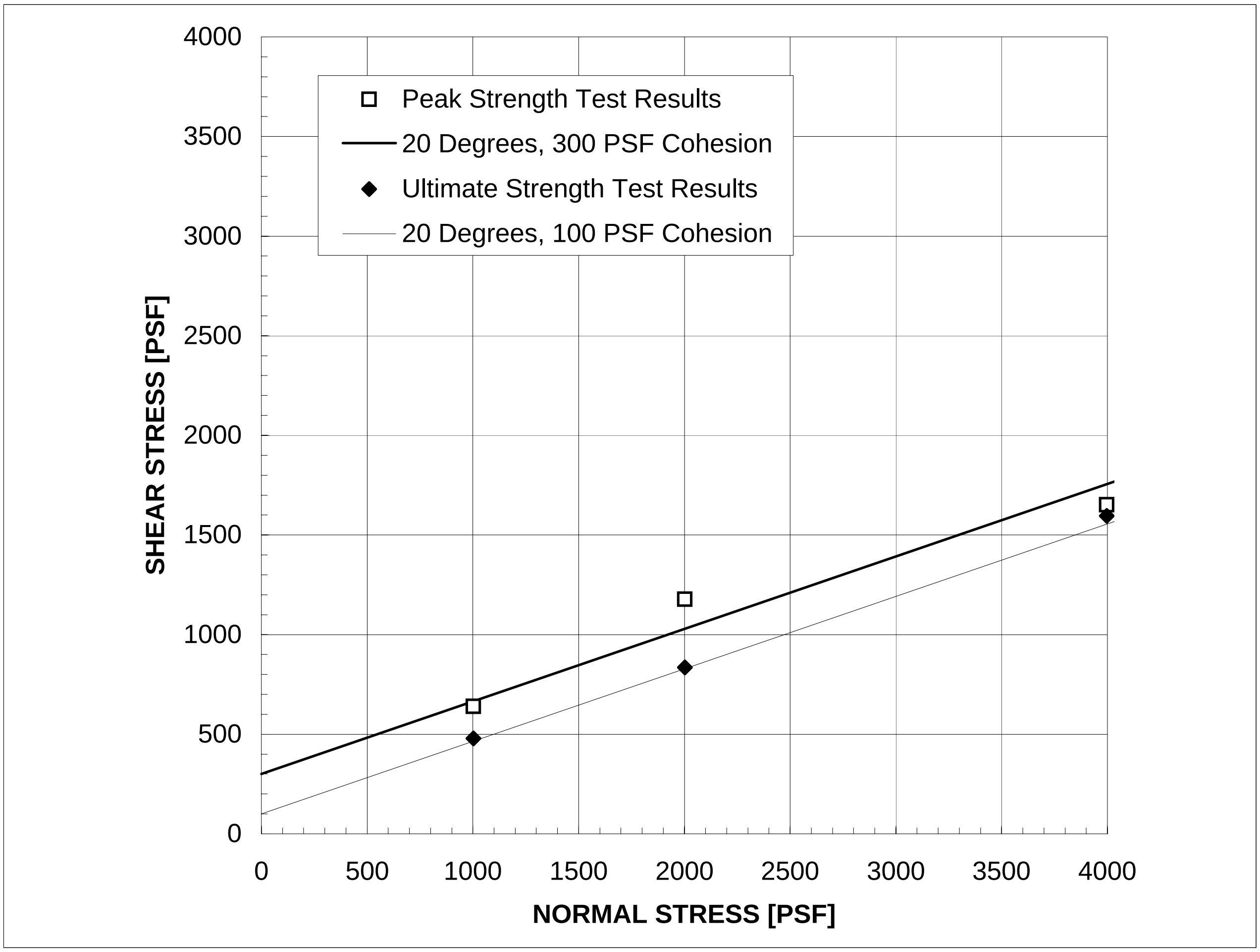
DIRECT SHEAR TEST RESULTS

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-5.2





SAMPLE: BH-12 @ 25' - 26'

<u>LACUSTRINE DEPOSITS:</u>
Dark brown fat clay (CH).

	PEAK	ULTIMATE
ϕ'	20 °	20 °
C'	300 PSF	100 PSF

STRAIN RATE: 0.0002 IN/MIN

(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	106.4 PCF	106.4 PCF
w_c	20.3 %	22.9 %

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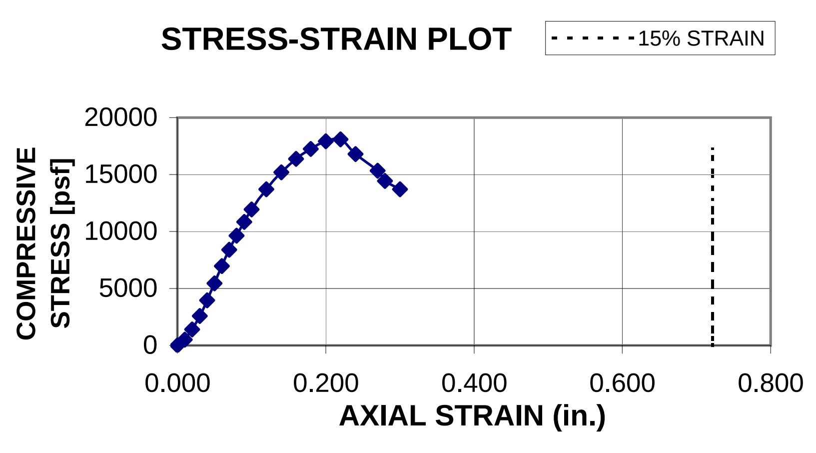
DIRECT SHEAR TEST RESULTS

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-5.3

PROJECT:	Niland Power Plant	SAMPLED BY:	JSO
SAMPLE I.D.:	BH-13 @ 20' - 21'	TESTED BY:	CAE
DESCRIPTION:	Dark brown fat clay (CH)	DATE:	6-Feb-06

TYPE OF SAMPLE	CAL	
WET WT. OF SAMPLE	727.85	[g]
INITIAL DIAM.	2.375	[in]
INITIAL HEIGHT	4.81	[in]
INITIAL AREA	4.4	[in^2]
INITIAL VOLUME	21.3	[in^3]
WET DENSITY	130.1	[pcf]
DRY WT. OF SAMPLE	618.16	[g]
WEIGHT OF WATER	109.7	[g]
MOISTURE CONTENT	17.7	[%]
DRY DENSITY	110.5	[pcf]
L-D RATIO	2.0:1	
STRAIN RATE	0.019	[in/min]
STRAIN AT FAILURE	3.33	[%]
STRAIN AT FAILURE	0.160	[in]
15% STRAIN	0.722	
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	17246	[psf]
SHEAR STRENGTH:	8623	[psf]
SPEC. GRAVITY	2.8	
by test: ☐		
estimate: ☒		
SATURATION:	86	[%]
FAILURE MODE:	Brittle	



SPECIMEN AFTER FAILURE

Elapsed Time [min]	**Axial Load** [lb]	**Strain Dial** [in]	**Total Strain** [in]	**Unit Strain**	**Corrected Area** [in^2]	**Stress** [psf]
0.0	0.0	1.000	0.000	0.000	4.43	0.00
0.7	15.4	0.990	0.010	0.002	4.44	499.53
1.1	43.0	0.980	0.020	0.004	4.45	1391.89
1.5	80.0	0.970	0.030	0.006	4.46	2584.15
1.9	122.9	0.960	0.040	0.008	4.47	3961.60
2.4	169.4	0.950	0.050	0.010	4.48	5449.05
2.9	217.1	0.940	0.060	0.012	4.49	6968.73
3.5	262.0	0.930	0.070	0.015	4.50	8392.28
4.0	301.8	0.920	0.080	0.017	4.51	9646.74
4.6	340.0	0.910	0.090	0.019	4.51	10844.79
5.2	375.0	0.900	0.100	0.021	4.52	11935.83
6.3	432.9	0.880	0.120	0.025	4.54	13720.21
7.4	481.8	0.860	0.140	0.029	4.56	15204.91
8.5	521.2	0.840	0.160	0.033	4.58	16377.88
9.5	551.2	0.820	0.180	0.037	4.60	17246.08

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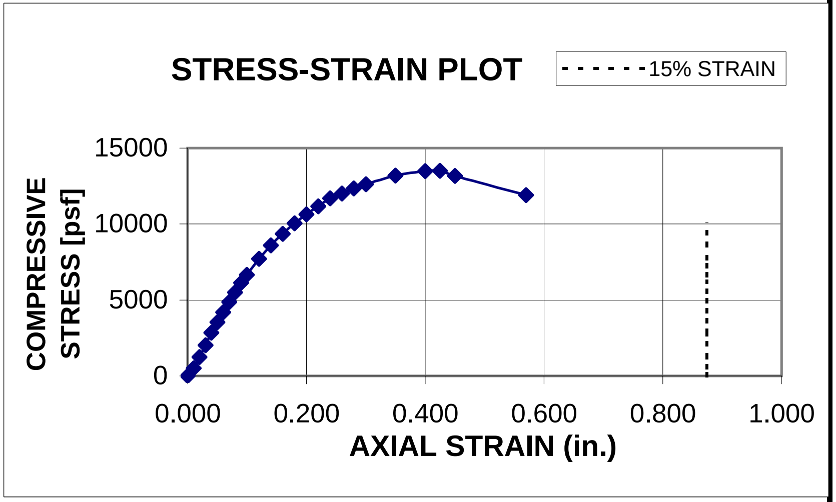
UNCONFINED COMPRESSIVE STRENGTH

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-6.1

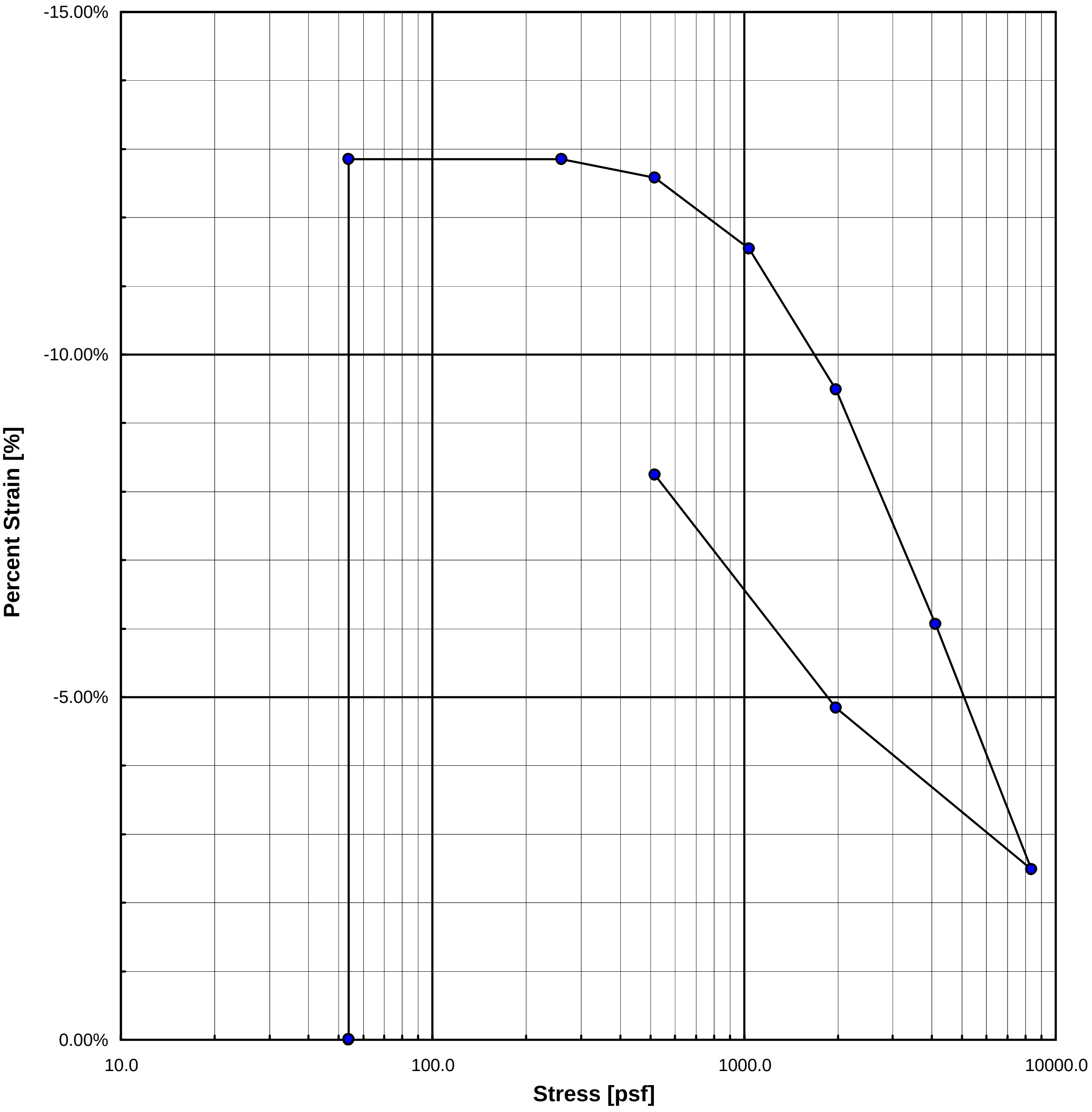
PROJECT:	Niland Power Plant	SAMPLED BY:	JSO
SAMPLE I.D.:	BH-13 @ 30' - 31'	TESTED BY:	CAE
DESCRIPTION:	Dark brown fat clay (CH)	DATE:	6-Feb-06

TYPE OF SAMPLE	CAL	
WET WT. OF SAMPLE	874.48	[g]
INITIAL DIAM.	2.375	[in]
INITIAL HEIGHT	5.83	[in]
INITIAL AREA	4.4	[in^2]
INITIAL VOLUME	25.8	[in^3]
WET DENSITY	129.0	[pcf]
DRY WT. OF SAMPLE	743.89	[g]
WEIGHT OF WATER	130.6	[g]
MOISTURE CONTENT	17.6	[%]
DRY DENSITY	109.7	[pcf]
L-D RATIO	2.5:1	
STRAIN RATE	0.022	[in/min]
STRAIN AT FAILURE	2.74	[%]
STRAIN AT FAILURE	0.160	[in]
15% STRAIN	0.875	
FAILURE CRITERIA:	Yield	
COMP. STRENGTH:	10049	[psf]
SHEAR STRENGTH:	5024	[psf]
SPEC. GRAVITY	2.8	
by test: ☐		
estimate: ☒		
SATURATION:	83	[%]
FAILURE MODE:	Brittle	



SPECIMEN AFTER FAILURE

Elapsed Time [min]	**Axial Load** [lb]	**Strain Dial** [in]	**Total Strain** [in]	**Unit Strain**	**Corrected Area** [in^2]	**Stress** [psf]
0.0	0.0	1.000	0.000	0.000	4.43	0.00
0.5	15.8	0.990	0.010	0.002	4.44	512.69
1.0	38.3	0.980	0.020	0.003	4.45	1240.66
1.3	62.6	0.970	0.030	0.005	4.45	2024.32
1.8	88.1	0.960	0.040	0.007	4.46	2844.01
2.3	110.0	0.950	0.050	0.009	4.47	3544.85
2.7	130.3	0.940	0.060	0.010	4.48	4191.77
3.1	151.4	0.930	0.070	0.012	4.48	4862.11
3.5	171.4	0.920	0.080	0.014	4.49	5494.85
4.1	191.3	0.910	0.090	0.015	4.50	6122.15
4.5	208.3	0.900	0.100	0.017	4.51	6654.58
5.5	242.3	0.880	0.120	0.021	4.52	7713.76
6.5	270.9	0.860	0.140	0.024	4.54	8594.05
7.3	295.9	0.840	0.160	0.027	4.56	9354.16
8.4	319.0	0.820	0.180	0.031	4.57	10048.84



BH-1 @ 20' - 21'

INITIAL	FINAL	
1.0000	1.0916	SAMPLE HEIGHT [IN]
107.0	98.0	DRY DENSITY [PCF]
2.88	2.88	SPECIFIC GRAVITY
0.68	0.83	VOID RATIO
21.6	29.0	WATER CONTENT [%]
91.2	100.2	DEGREE OF SATURATION [%]

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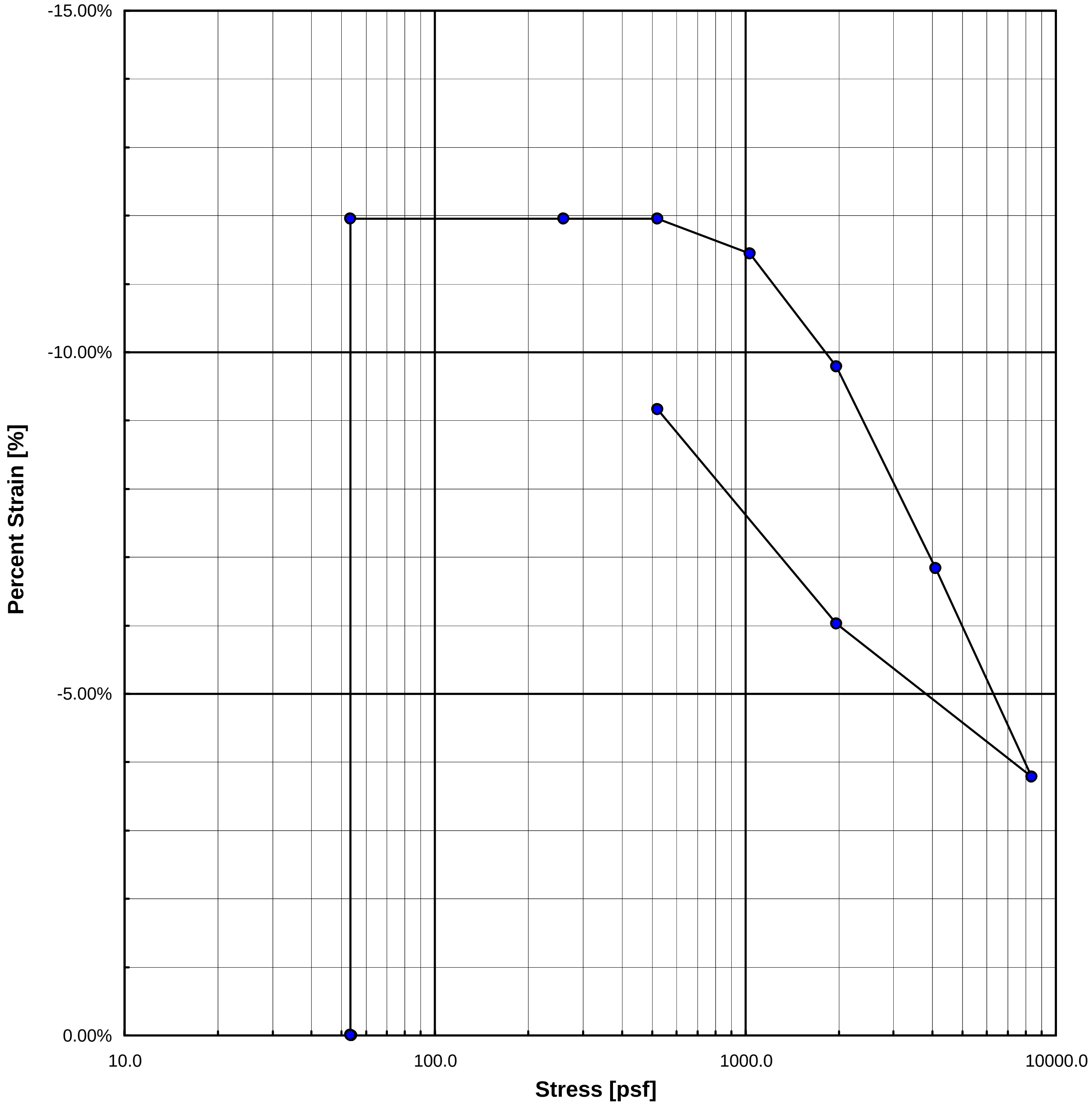
CONSOLIDATION TEST RESULTS

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-7.1



BH-1 @ 30' - 31'

INITIAL	FINAL	
1.0000	1.0910	SAMPLE HEIGHT [IN]
106.5	97.6	DRY DENSITY [PCF]
2.80	2.80	SPECIFIC GRAVITY
0.64	0.79	VOID RATIO
21.4	28.3	WATER CONTENT [%]
93.3	100.2	DEGREE OF SATURATION [%]

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CONSOLIDATION TEST RESULTS

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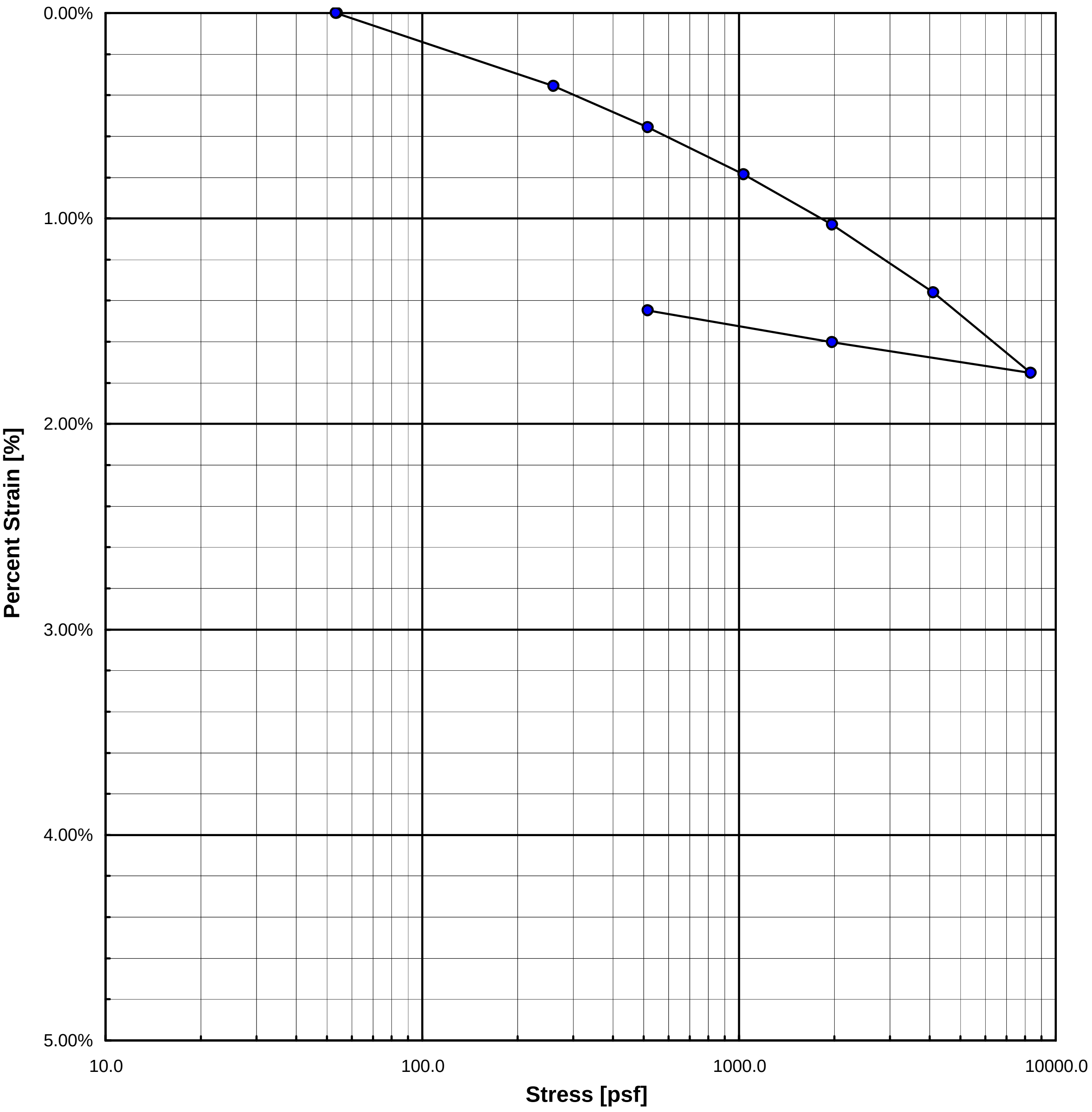
FIGURE E-7.2



BH-9 @ 10' - 11'

INITIAL	FINAL	
1.0000	0.9849	SAMPLE HEIGHT [IN]
111.9	113.6	DRY DENSITY [PCF]
2.80	2.80	SPECIFIC GRAVITY
0.56	0.54	VOID RATIO
18.0	17.4	WATER CONTENT [%]
89.6	90.3	DEGREE OF SATURATION [%]

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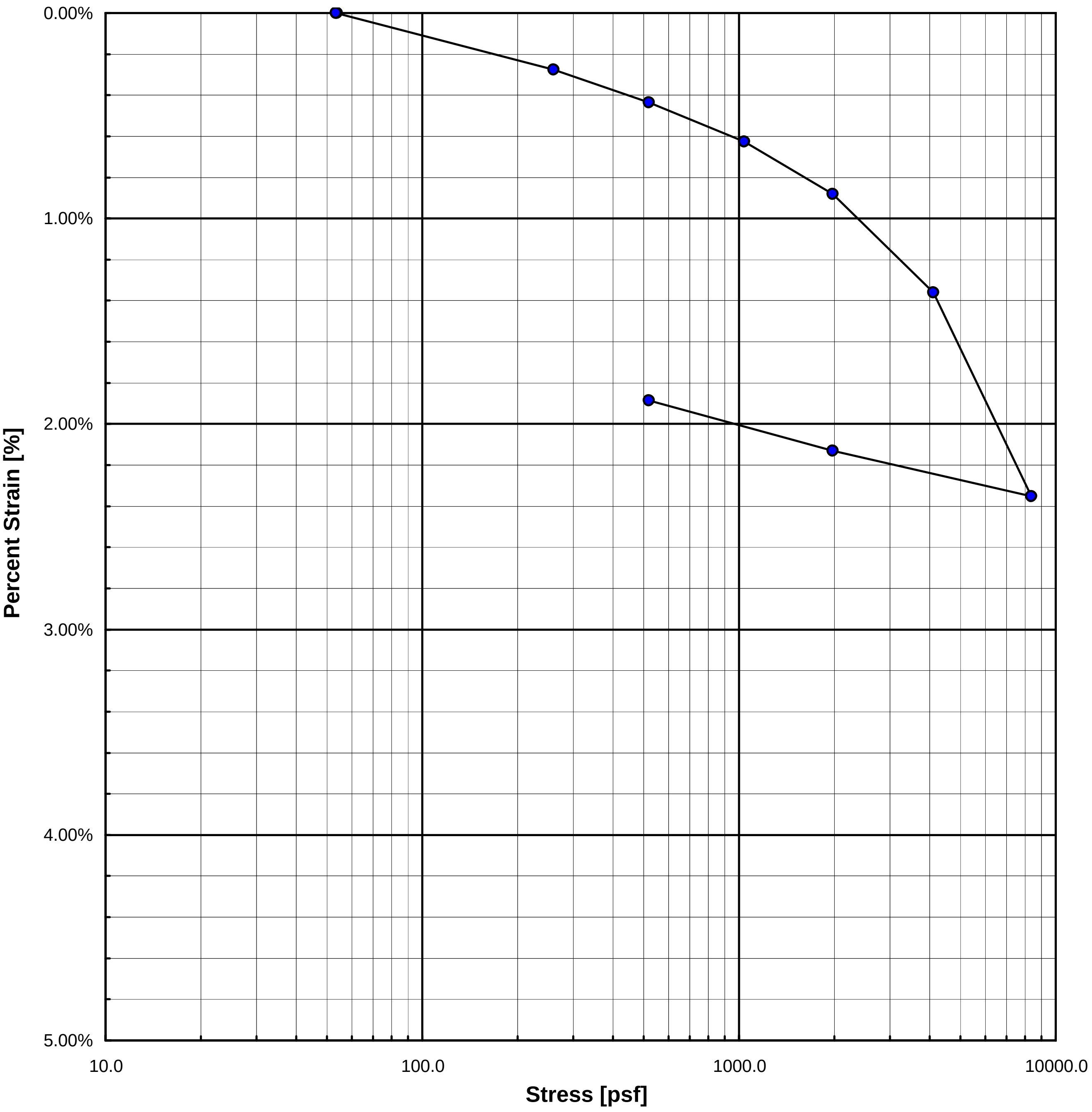
CONSOLIDATION TEST RESULTS

Project No. 0554-075-00
Document No. 06-0015
FIGURE E-7.3



BH-9 @ 20' - 21'

INITIAL	FINAL	
1.0000	0.9795	SAMPLE HEIGHT [IN]
109.2	111.4	DRY DENSITY [PCF]
2.80	2.80	SPECIFIC GRAVITY
0.60	0.57	VOID RATIO
20.0	19.6	WATER CONTENT [%]
93.0	96.4	DEGREE OF SATURATION [%]

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CONSOLIDATION TEST RESULTS

APPENDIX F

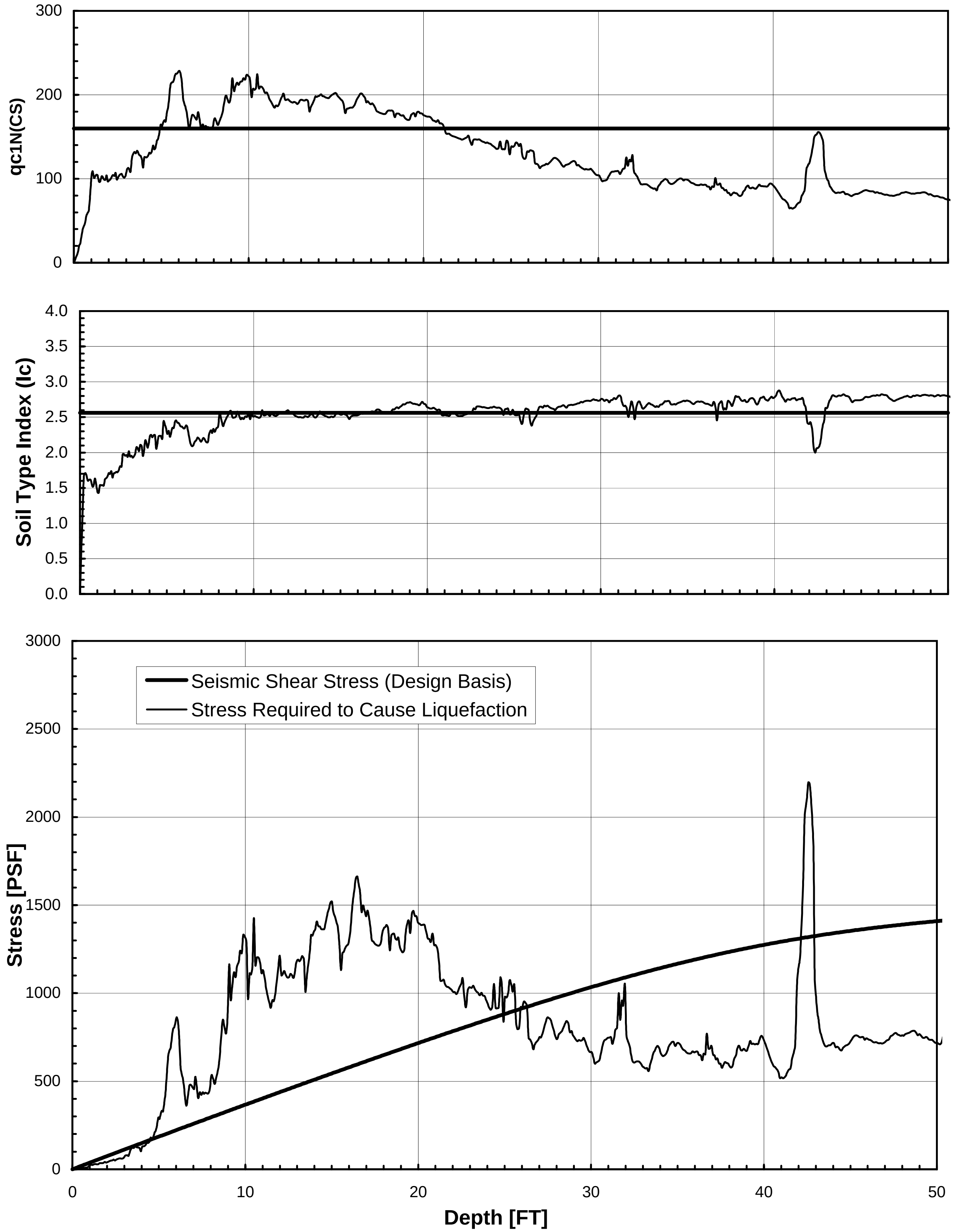
LIQUEFACTION ANALYSIS

Liquefaction analysis was performed on the data gathered from the CPT soundings. The liquefaction analysis was based on the simplified techniques originally presented by Seed and Idriss (1982), with recent improvements from the 1996 and 1998 NCEER workshops as summarized by Youd (2001). The liquefaction analysis was conducted in general accordance with the recommended procedures for implementation of DMG special publication 117 (SCEC, 1999). The CPT data $(q_{c1N})_{cs}$ was normalized for overburden pressure, and corrected for fines content using the methods described in the referenced document (Youd, 2001). The CPT fines correction was based on the Soil Behavior Type Index (Ic). The results of the liquefaction analyses are presented in the following Figures F-1.1 through F-6.4.

The first figure for each CPT sounding (Figures F-1.1 to F-6.1) presents an overview of the soil density, soil type, and liquefaction potential. The bottom chart shows the stress required to cause liquefaction versus the stress induced by the *upper bound* magnitude weighted peak ground acceleration (0.48g). At depths where the seismic stress exceeds the stress required to cause liquefaction, the potential may exist for liquefaction. The middle chart shows the Soil Behavior Type Index (Ic) plotted as a function of depth. Note that soils with an Ic value greater than 2.6 are generally considered too clayey to liquefy (soils with a slightly lower Ic value may also be too clayey to liquefy). The top chart shows the normalized clean sand equivalent tip resistance $(q_{c1N})_{cs}$ plotted as a function of depth. Note that sandy soils with a $(q_{c1N})_{cs}$ value greater than 160 may be too dense to liquefy.

The second figure for each CPT sounding presents the same charts described above (Figures F-1.2 through F-6.2). However, unsaturated soil zones are excluded from the bottom chart. None of the soils observed on site are considered to be liquefiable at the present time, because no groundwater was encountered in our subsurface explorations.

The next figure presents an estimate of the seismic settlement at each CPT sounding location (Figures F-1.3 through F-6.3). Seismic settlement of unsaturated soil with a $(q_{c1N})_{cs}$ value less than 200 is included in the settlement estimate. Only those soil zones which may be sandy enough to liquefy contribute to the estimated seismic settlement. The final figure for each CPT sounding presents an estimate of the settlement at each CPT sounding location after excavation and compaction of the upper 4 feet of soil (Figures F-1.4 through F-6.4). Note that the recommended 4 foot thick compacted fill substantially reduces the estimated seismic settlement.



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LIQUEFACTION ANALYSIS (CPT-2)

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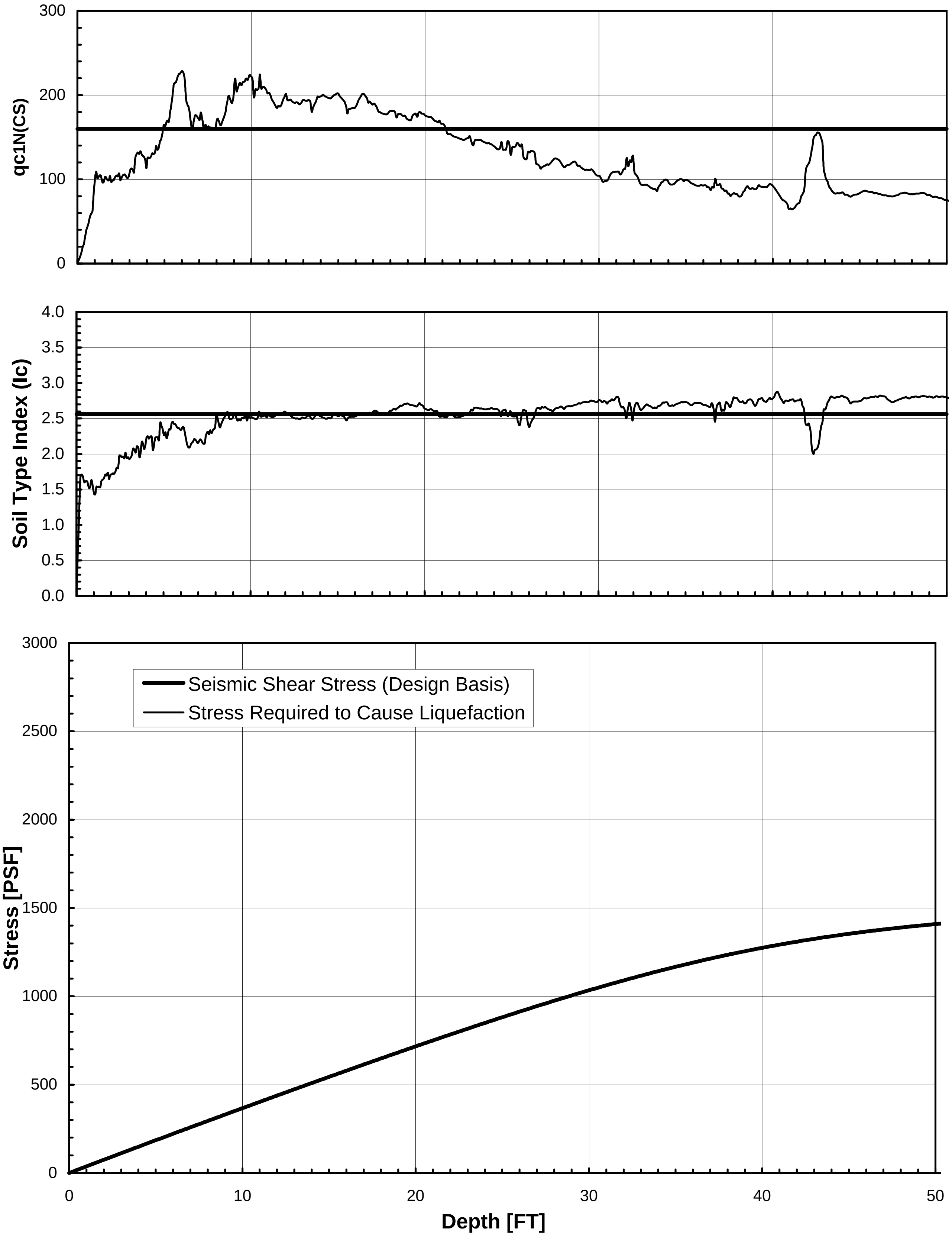
Document No. 06-0015
FIGURE F-1.1



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LIQUEFACTION ANALYSIS (CPT-2)

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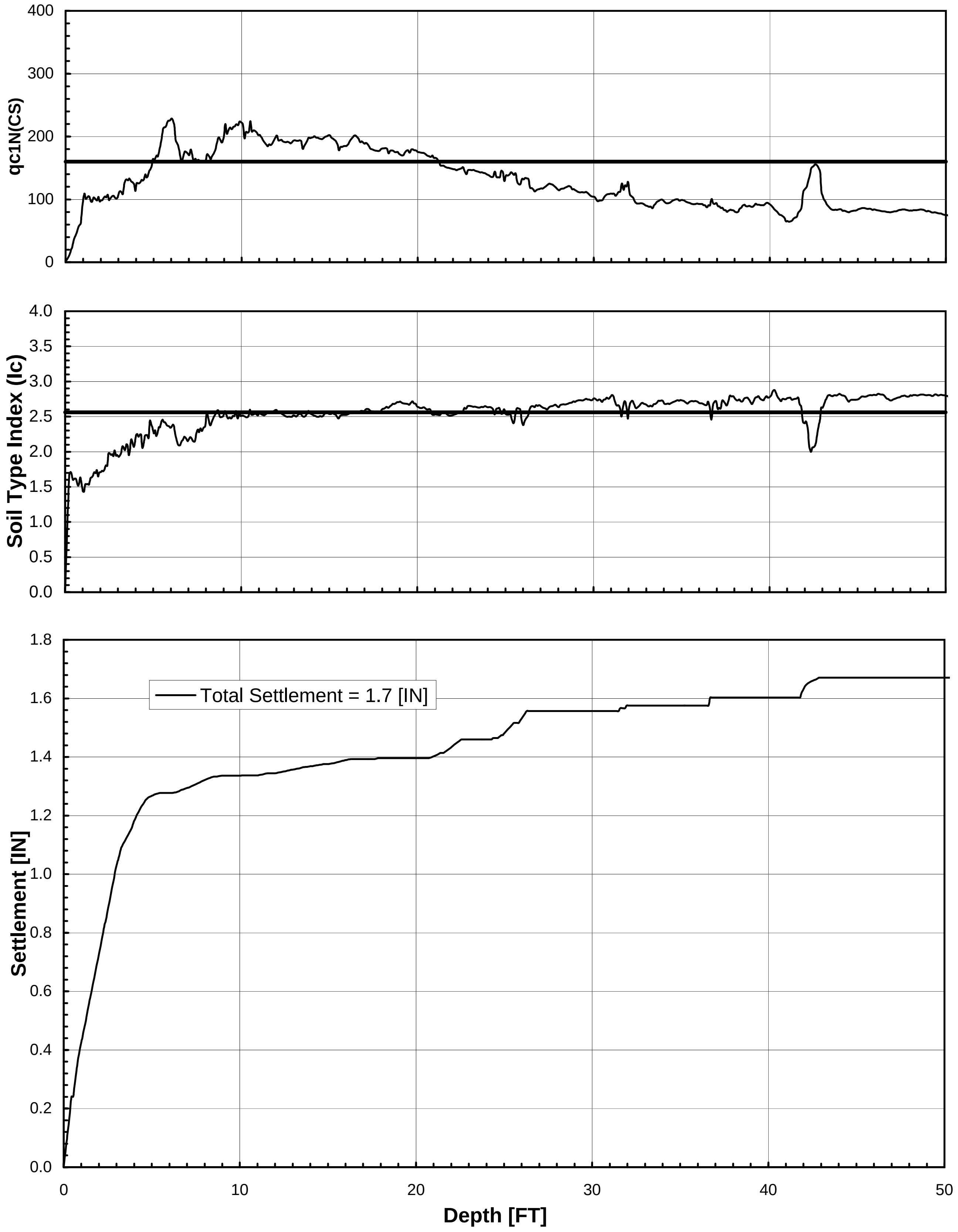
Document No. 06-0015
FIGURE F-1.2



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SETTLEMENT ANALYSIS (CPT-2)

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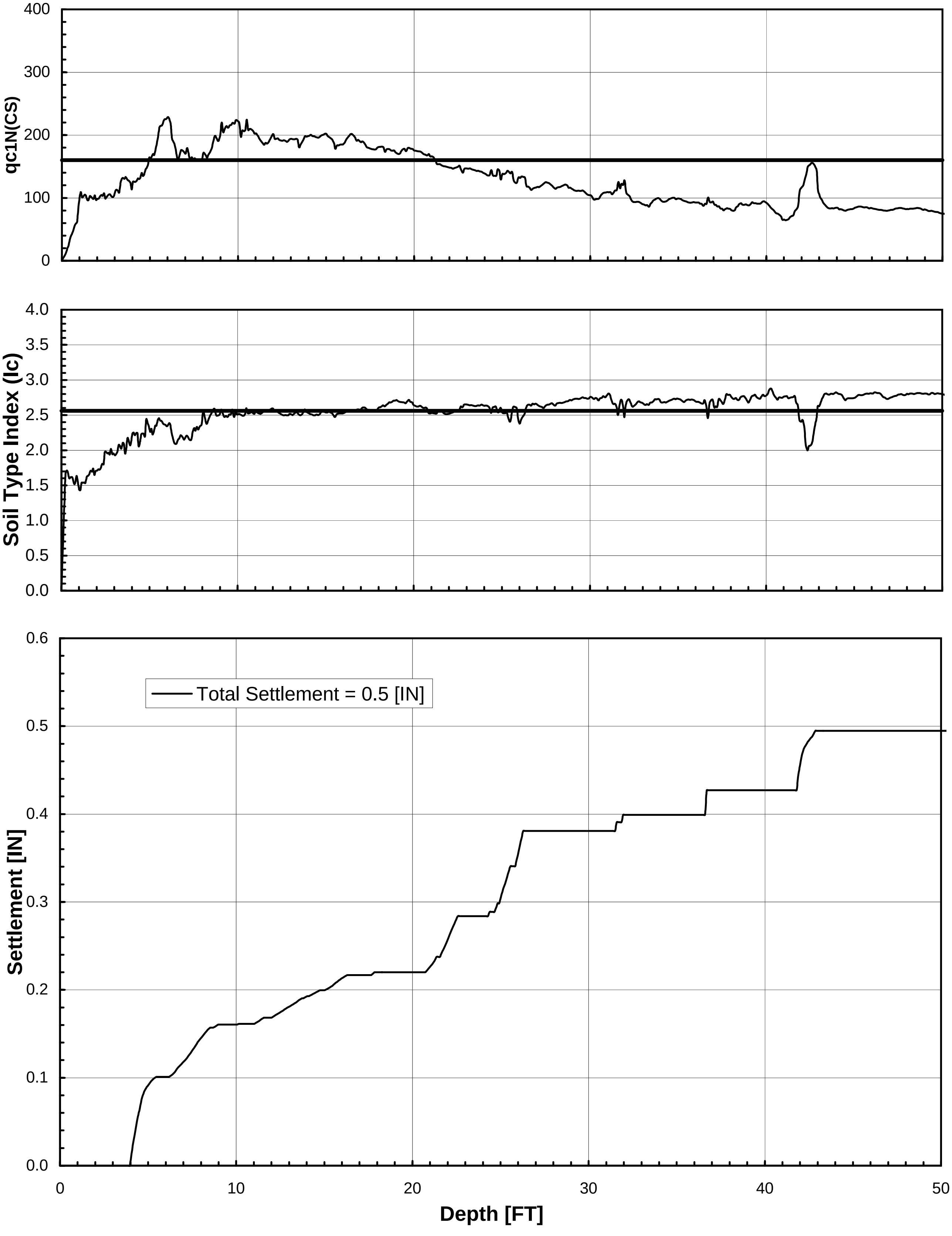
Document No. 06-0015
FIGURE F-1.3



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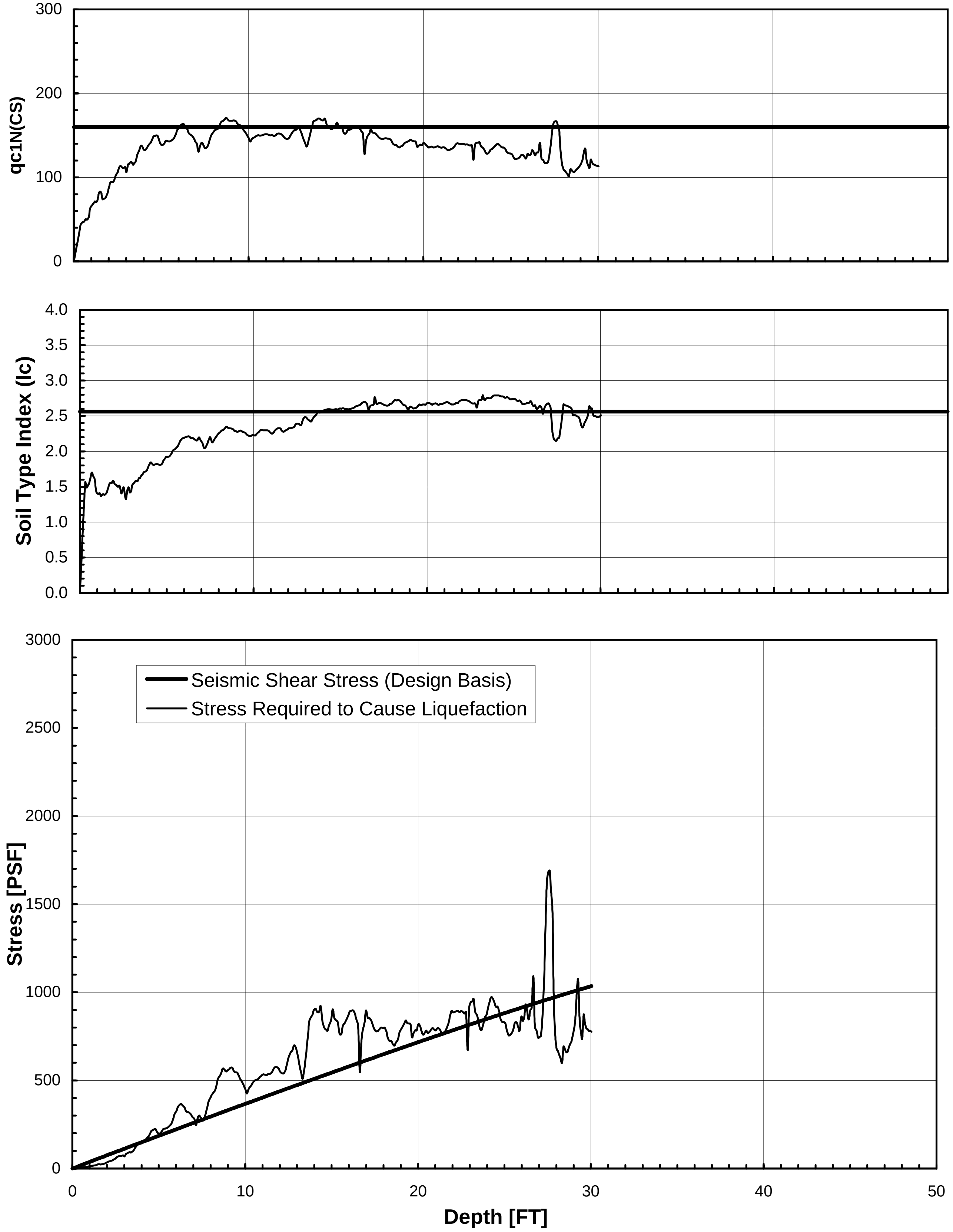
SETTLEMENT ANALYSIS (CPT-2)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-1.4



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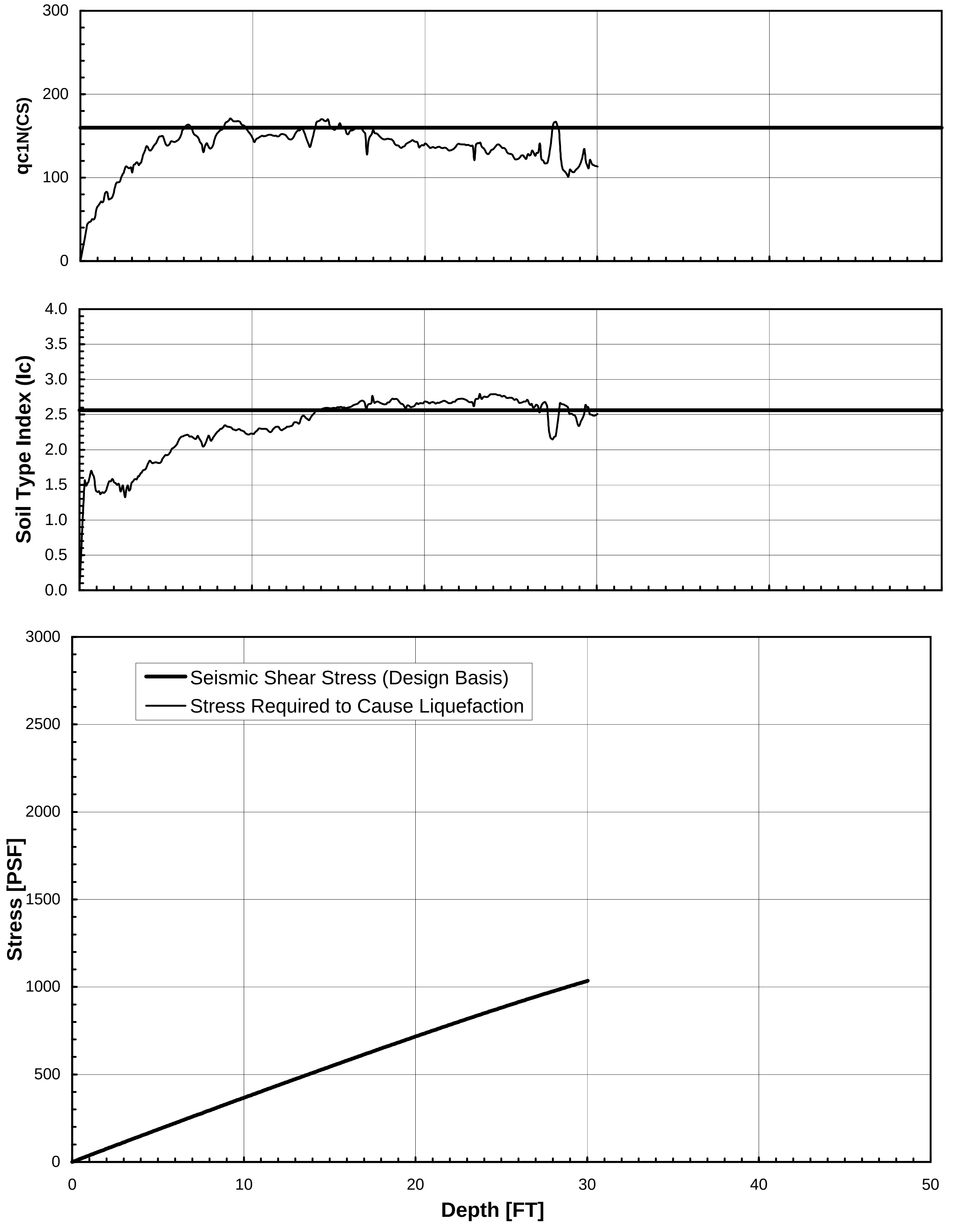
LIQUEFACTION ANALYSIS (CPT-7)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-2.1



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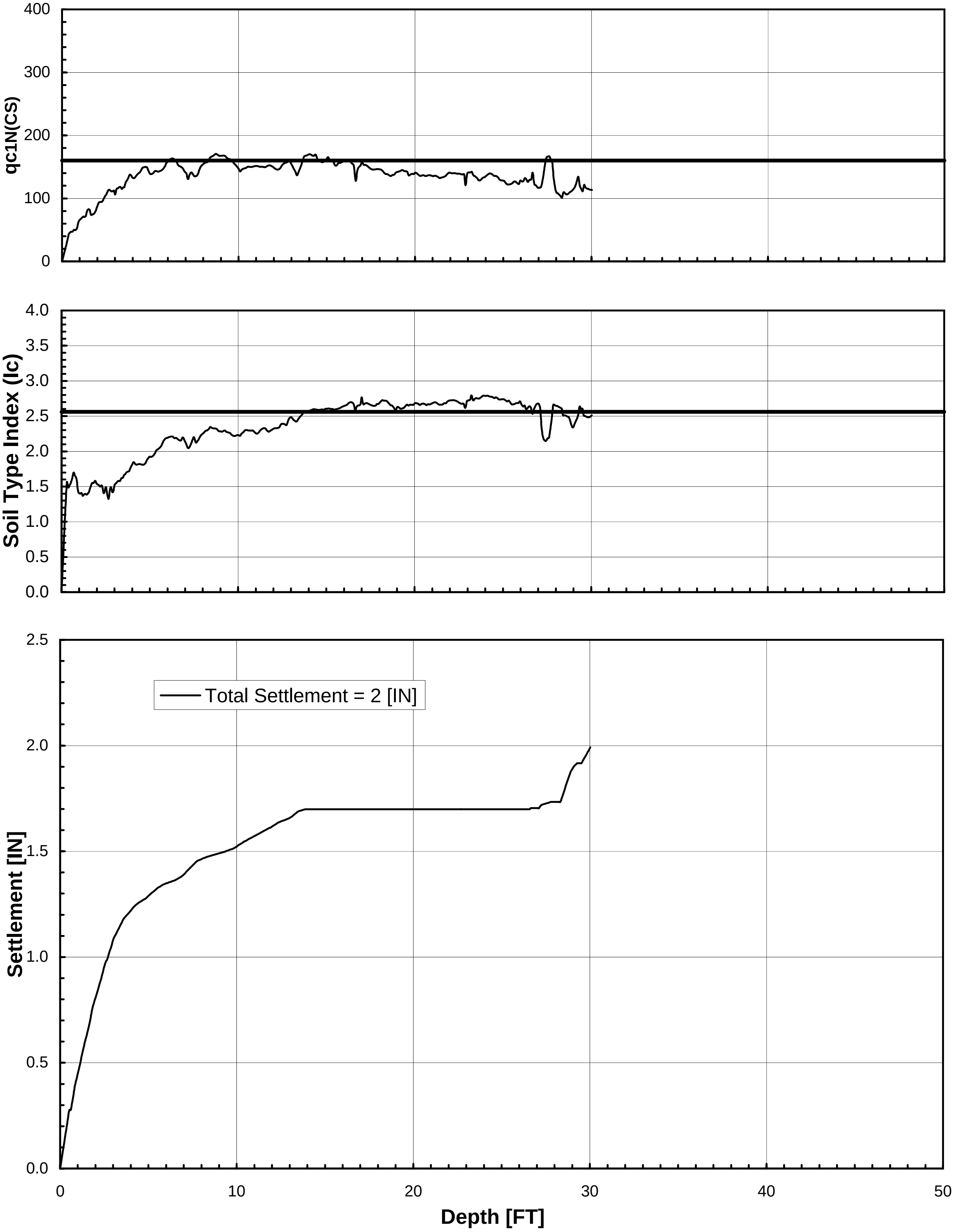
LIQUEFACTION ANALYSIS (CPT-7)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-2.2



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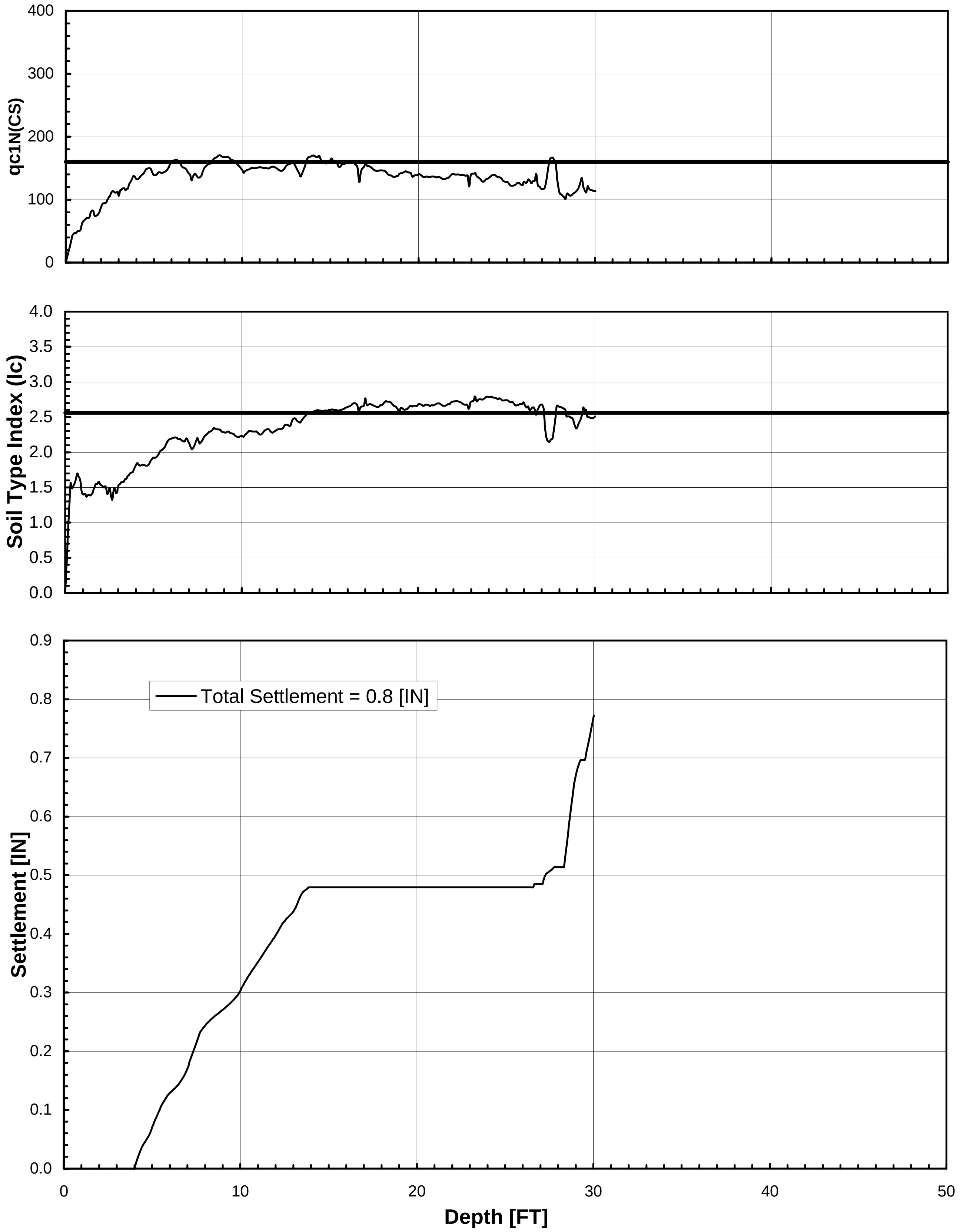
SETTLEMENT ANALYSIS (CPT-7)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-2.3



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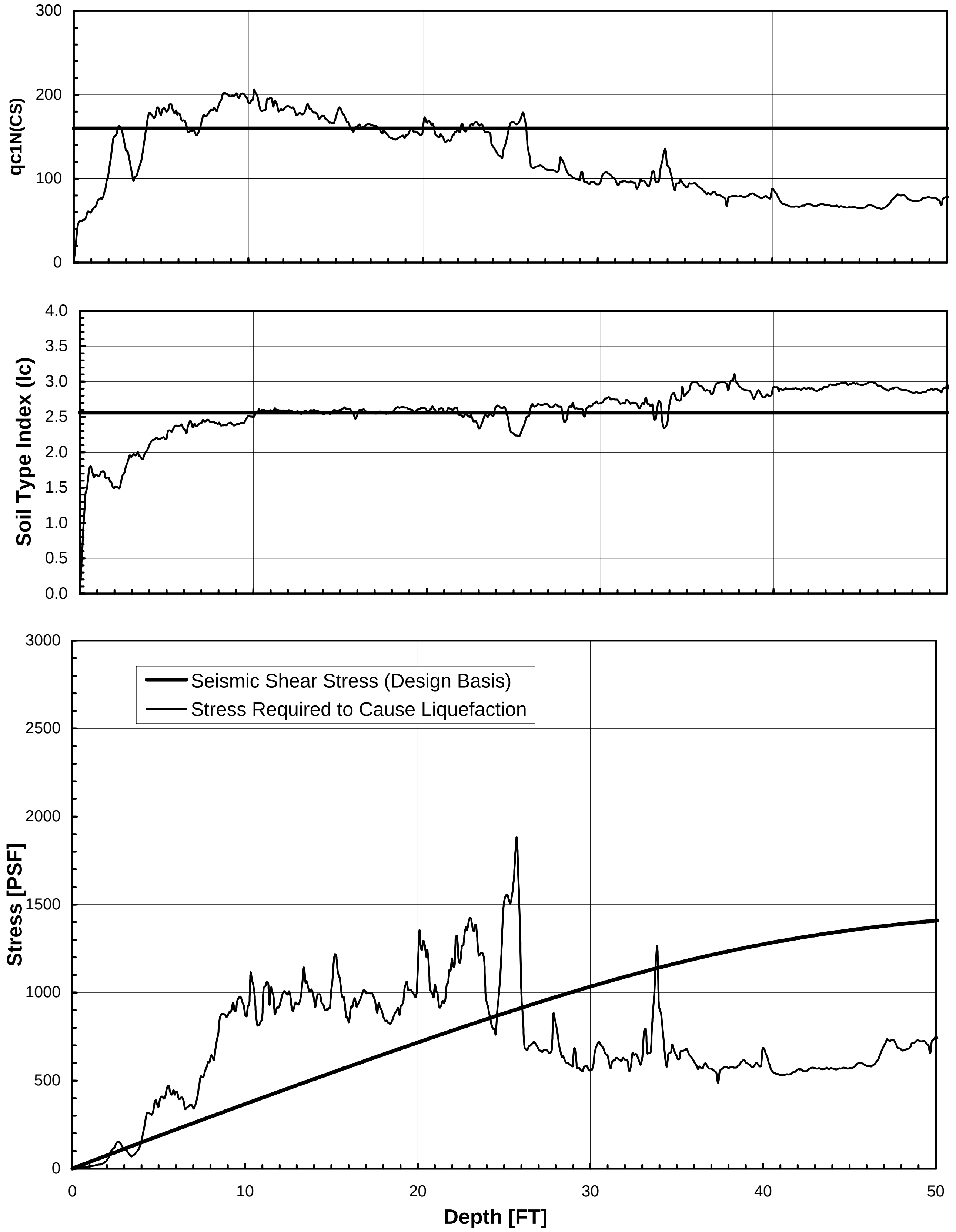
SETTLEMENT ANALYSIS (CPT-7)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-2.4



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LIQUEFACTION ANALYSIS (CPT-10)

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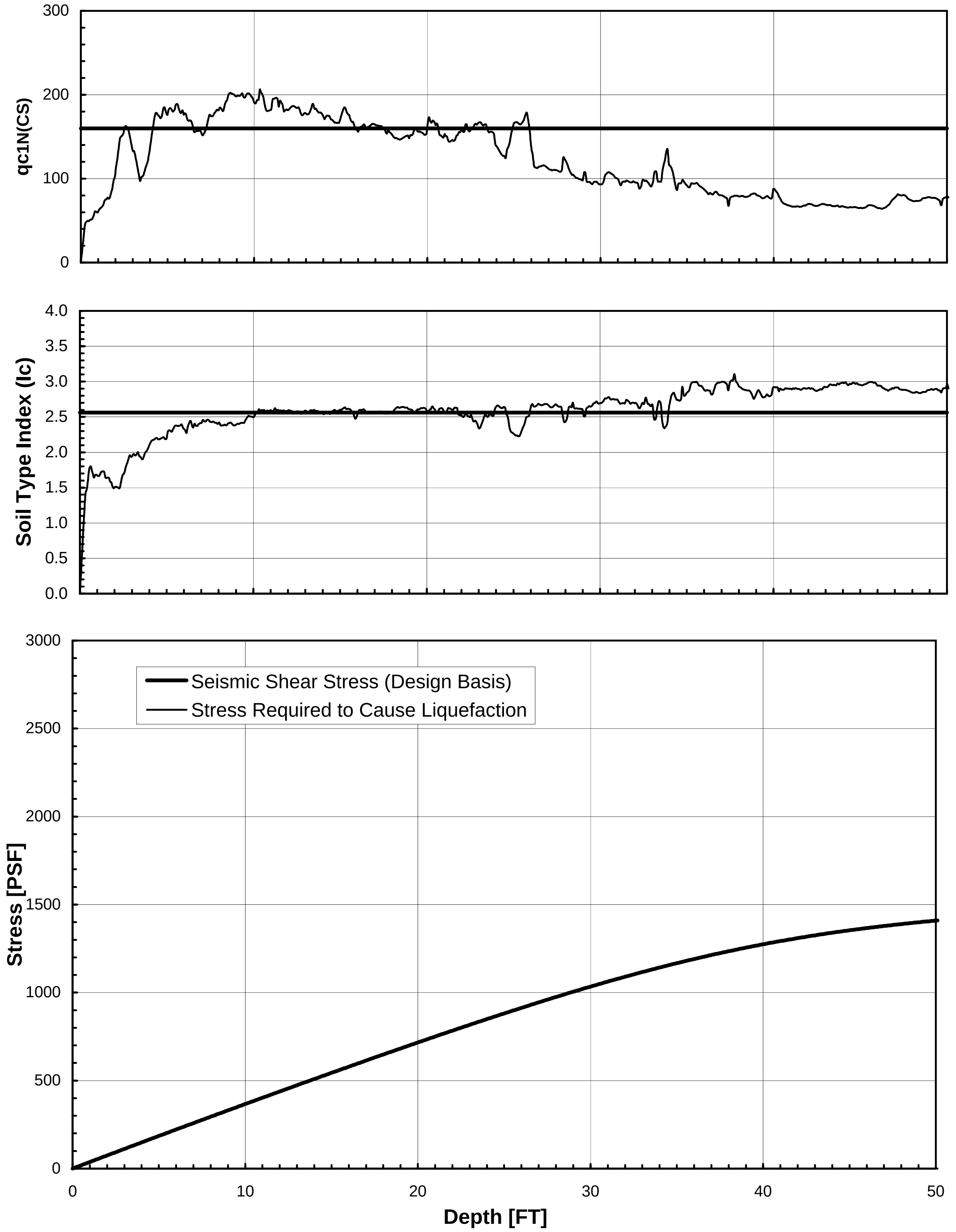
Document No. 06-0015
FIGURE F-3.1



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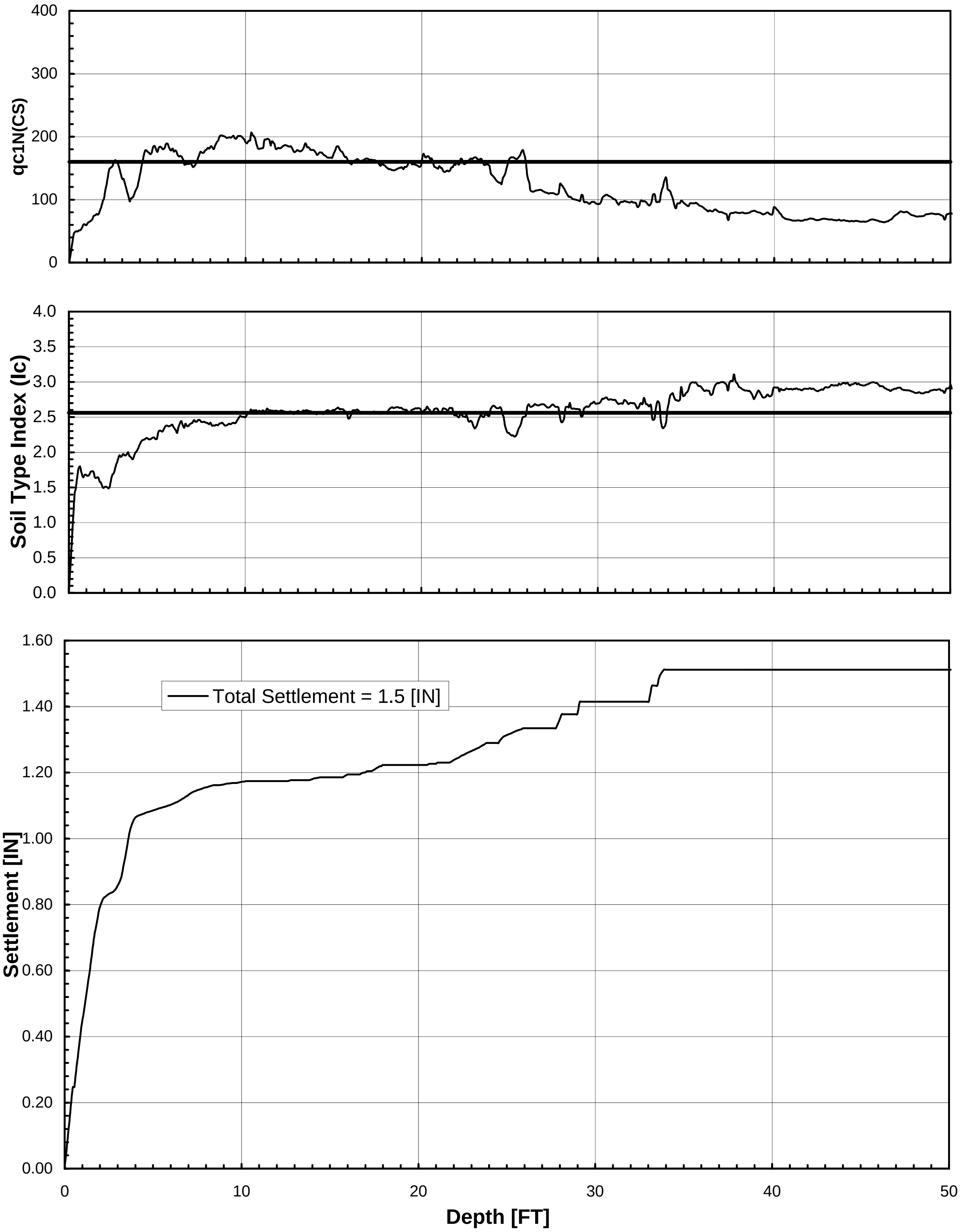
LIQUEFACTION ANALYSIS (CPT-10)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-3.2



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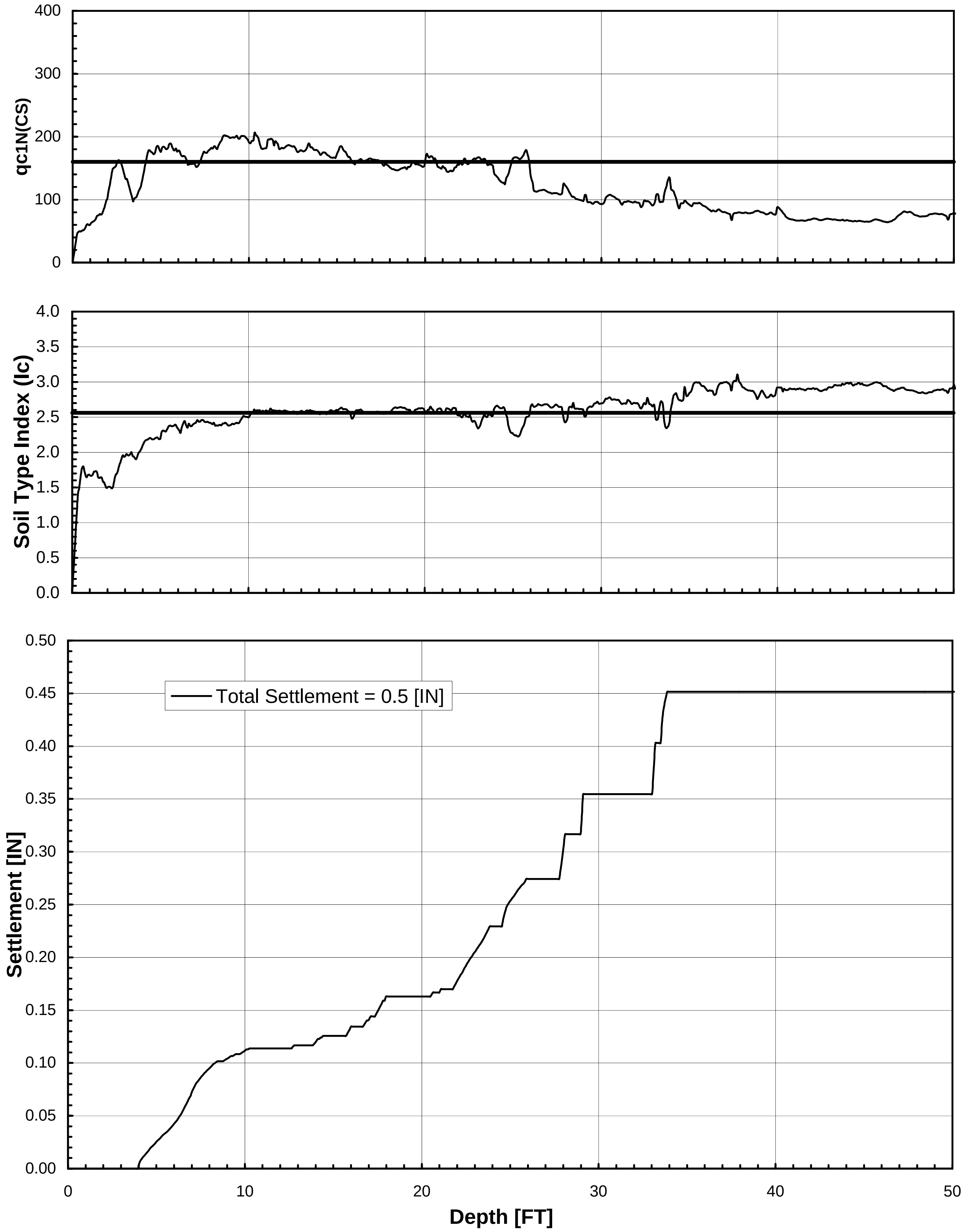
SETTLEMENT ANALYSIS (CPT-10)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-3.3



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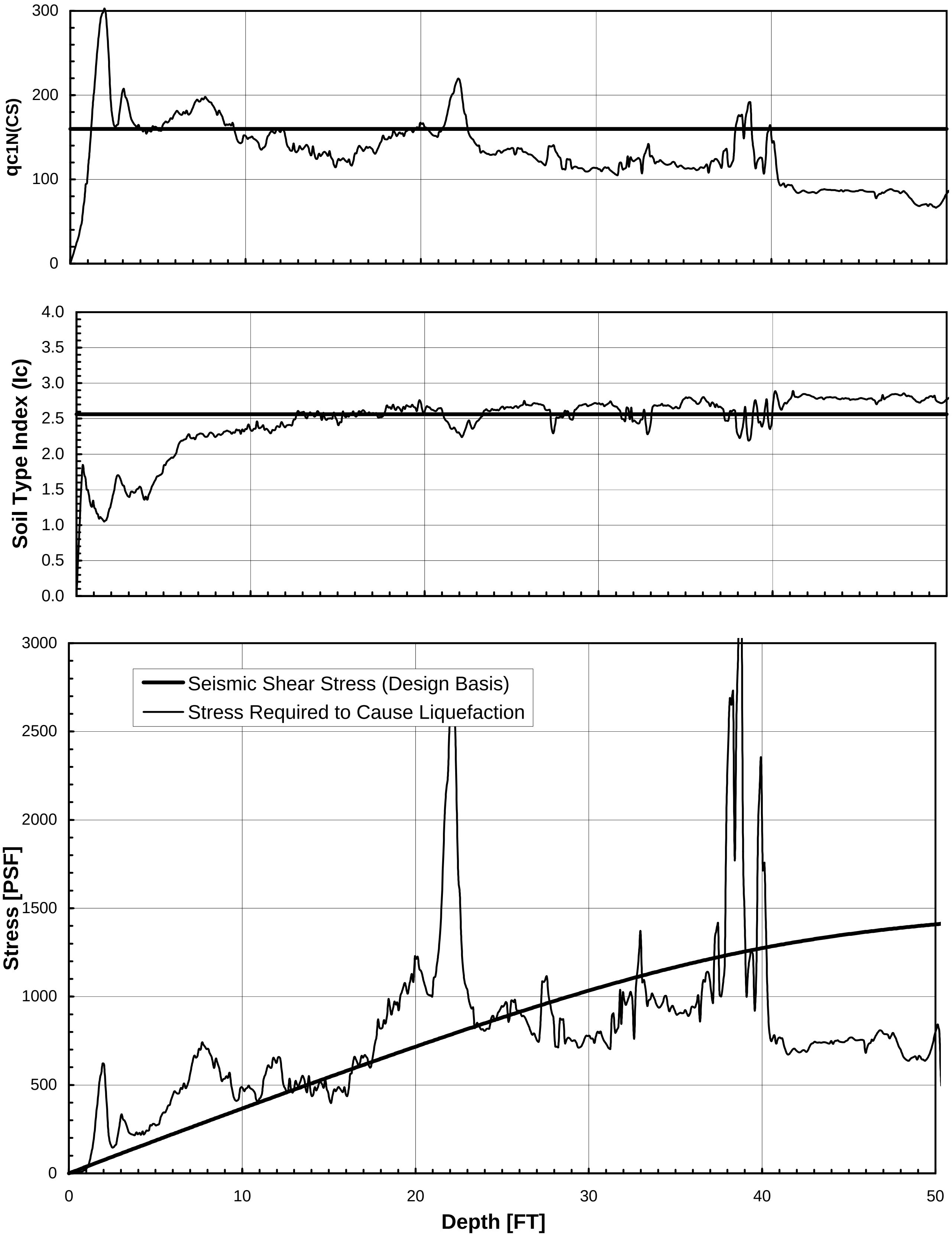
SETTLEMENT ANALYSIS (CPT-10)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-3.4



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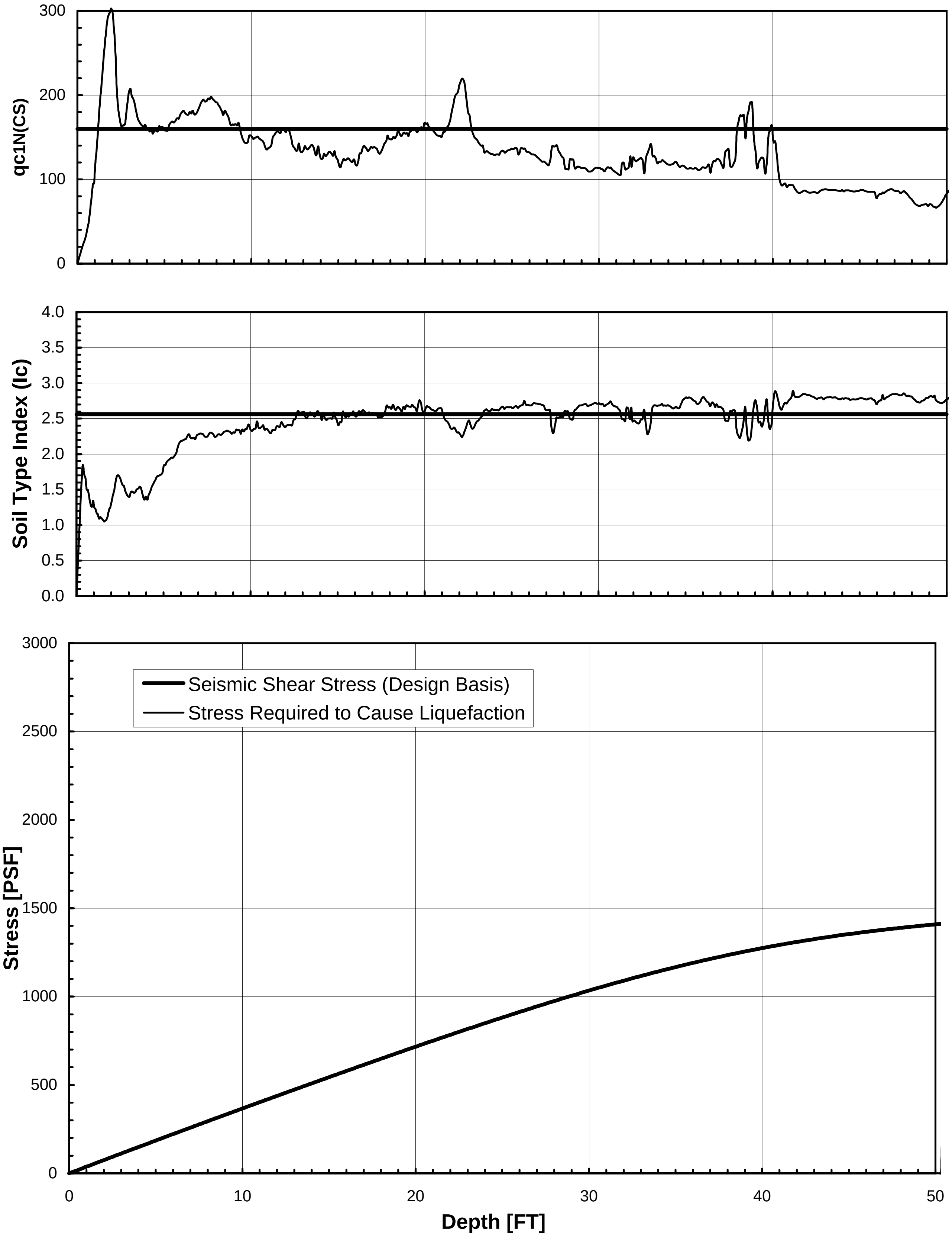
LIQUEFACTION ANALYSIS (CPT-11)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-4.1



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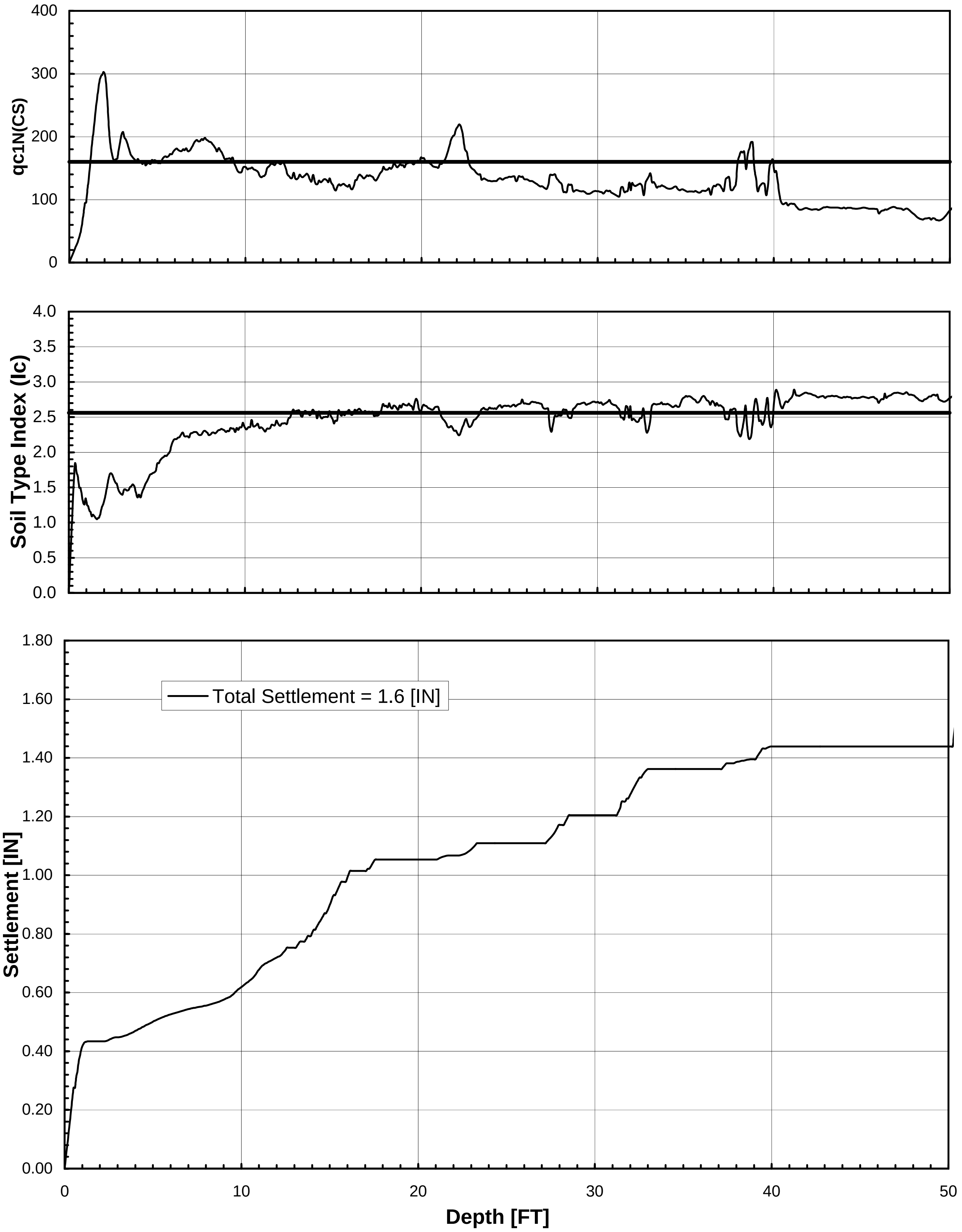
LIQUEFACTION ANALYSIS (CPT-11)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-4.2



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SETTLEMENT ANALYSIS (CPT-11)

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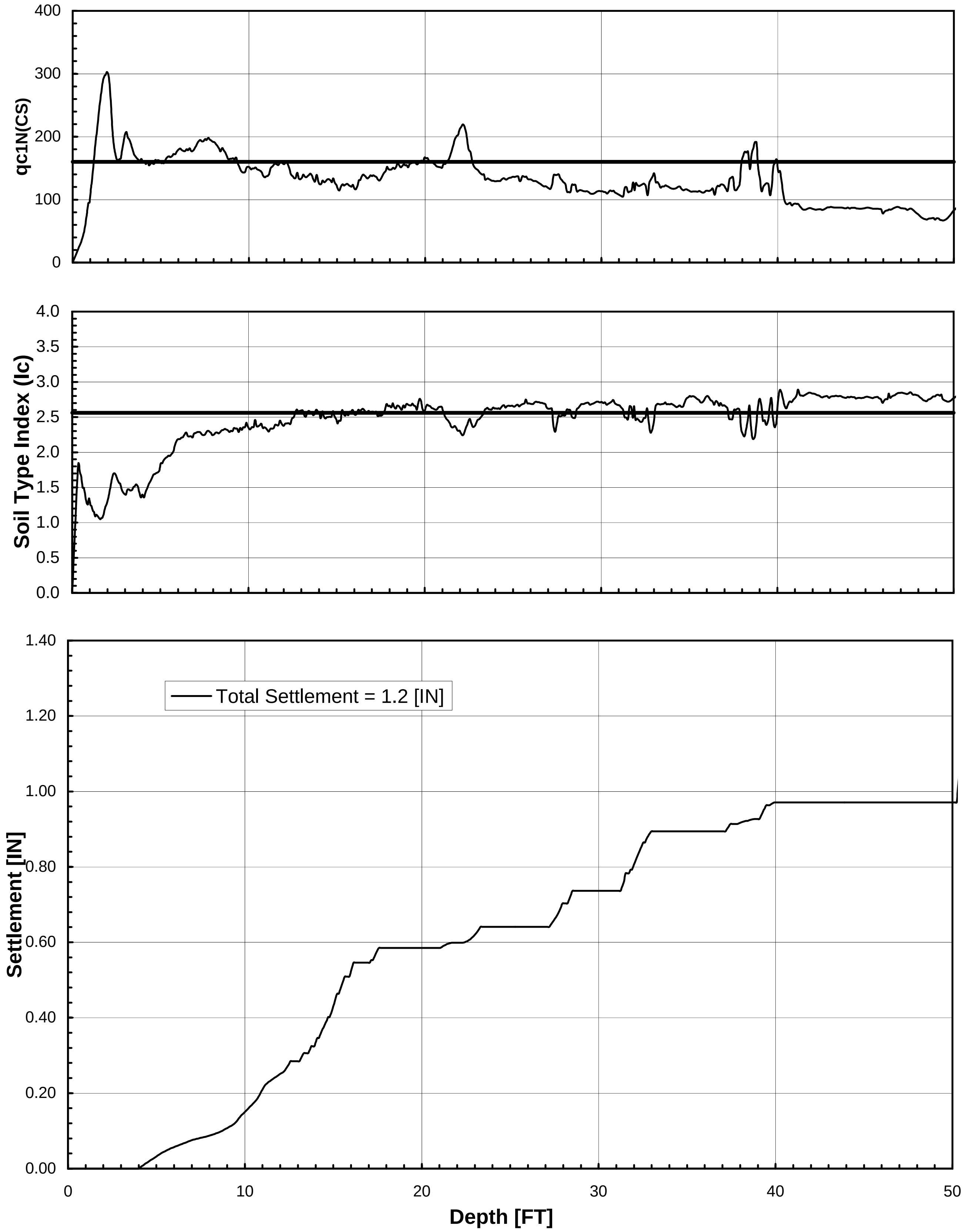
Document No. 06-0015
FIGURE F-4.3



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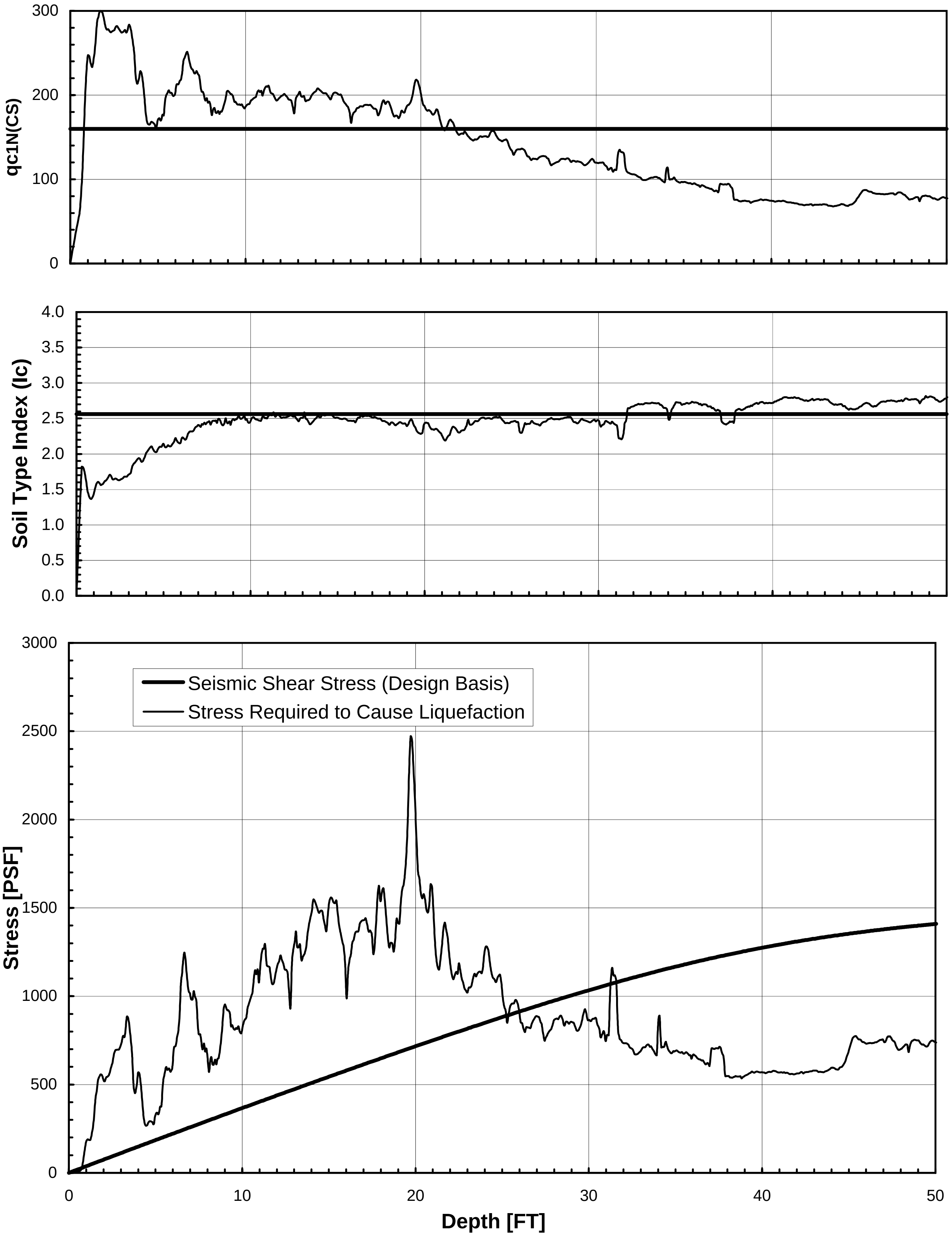
SETTLEMENT ANALYSIS (CPT-11)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-4.4



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LIQUEFACTION ANALYSIS (CPT-14)

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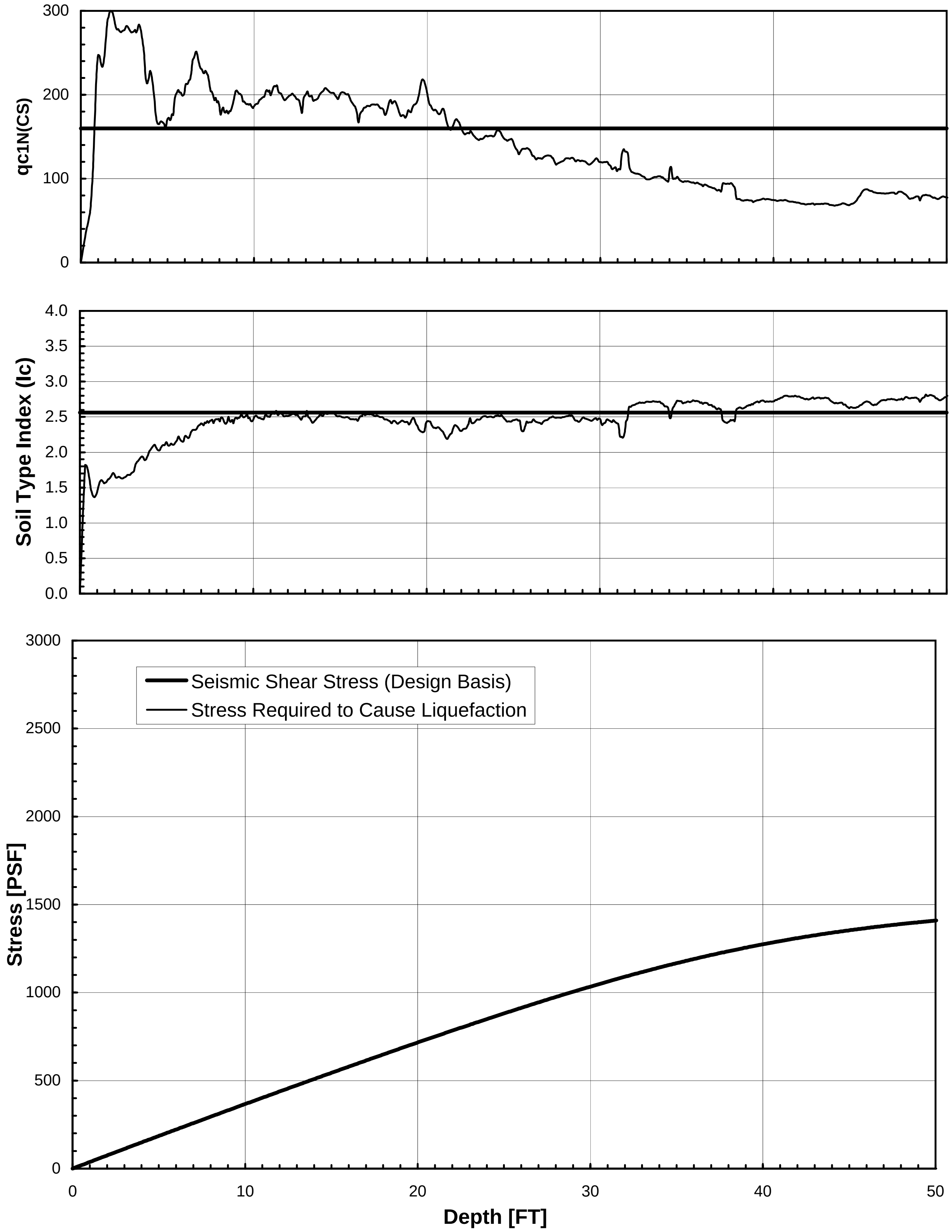
Document No. 06-0015
FIGURE F-5.1



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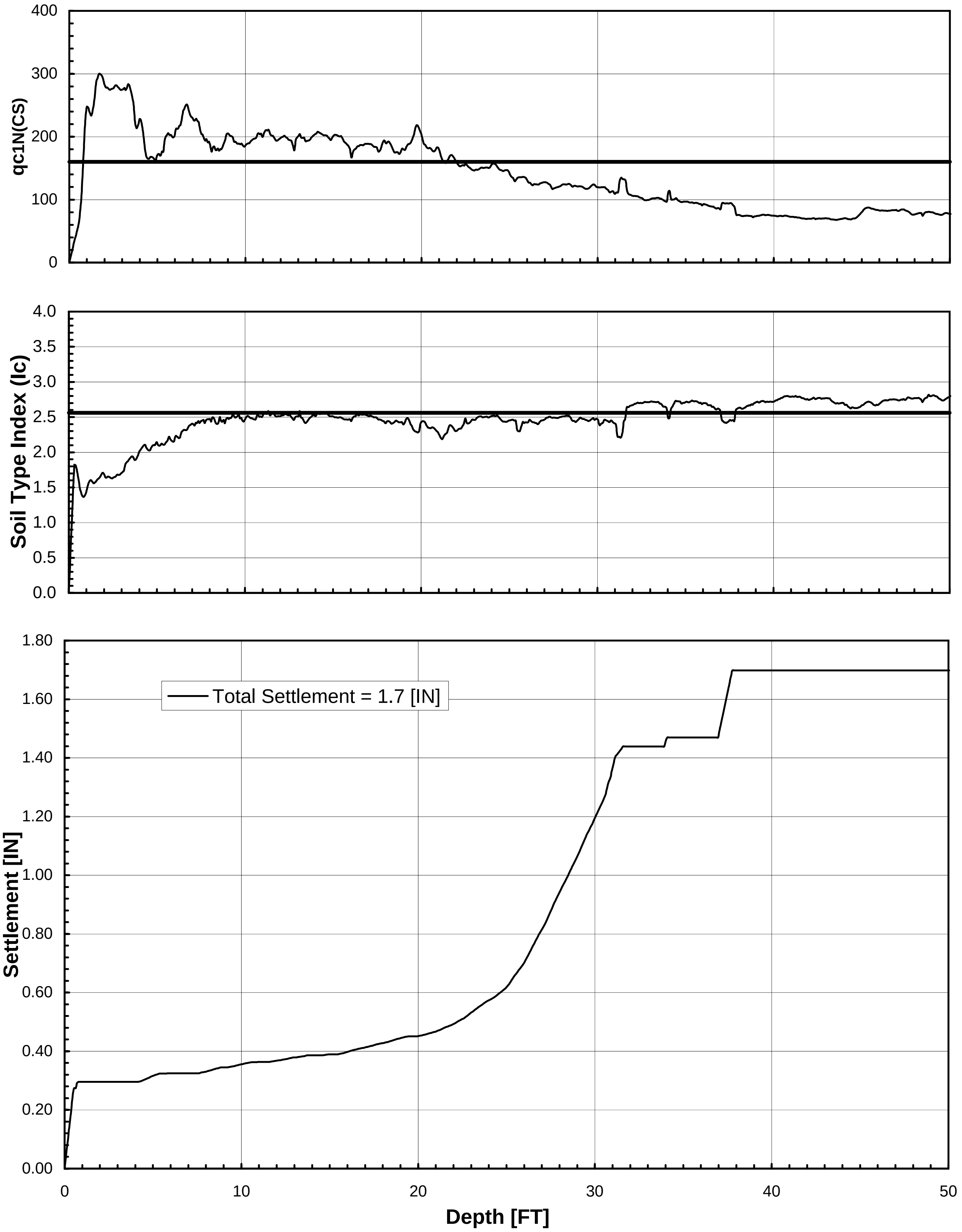
LIQUEFACTION ANALYSIS (CPT-14)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-5.2



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SETTLEMENT ANALYSIS (CPT-14)

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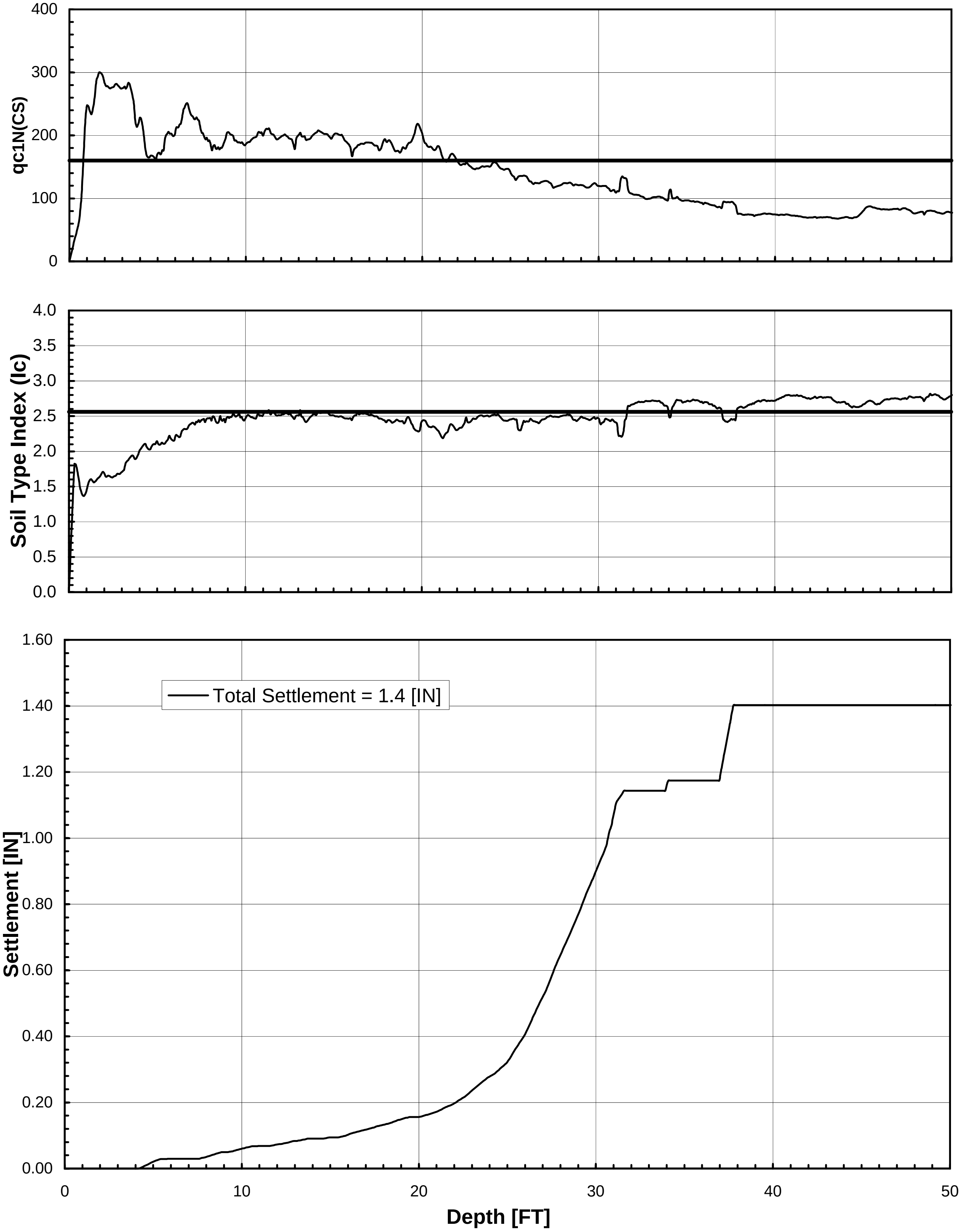
Document No. 06-0015
FIGURE F-5.3



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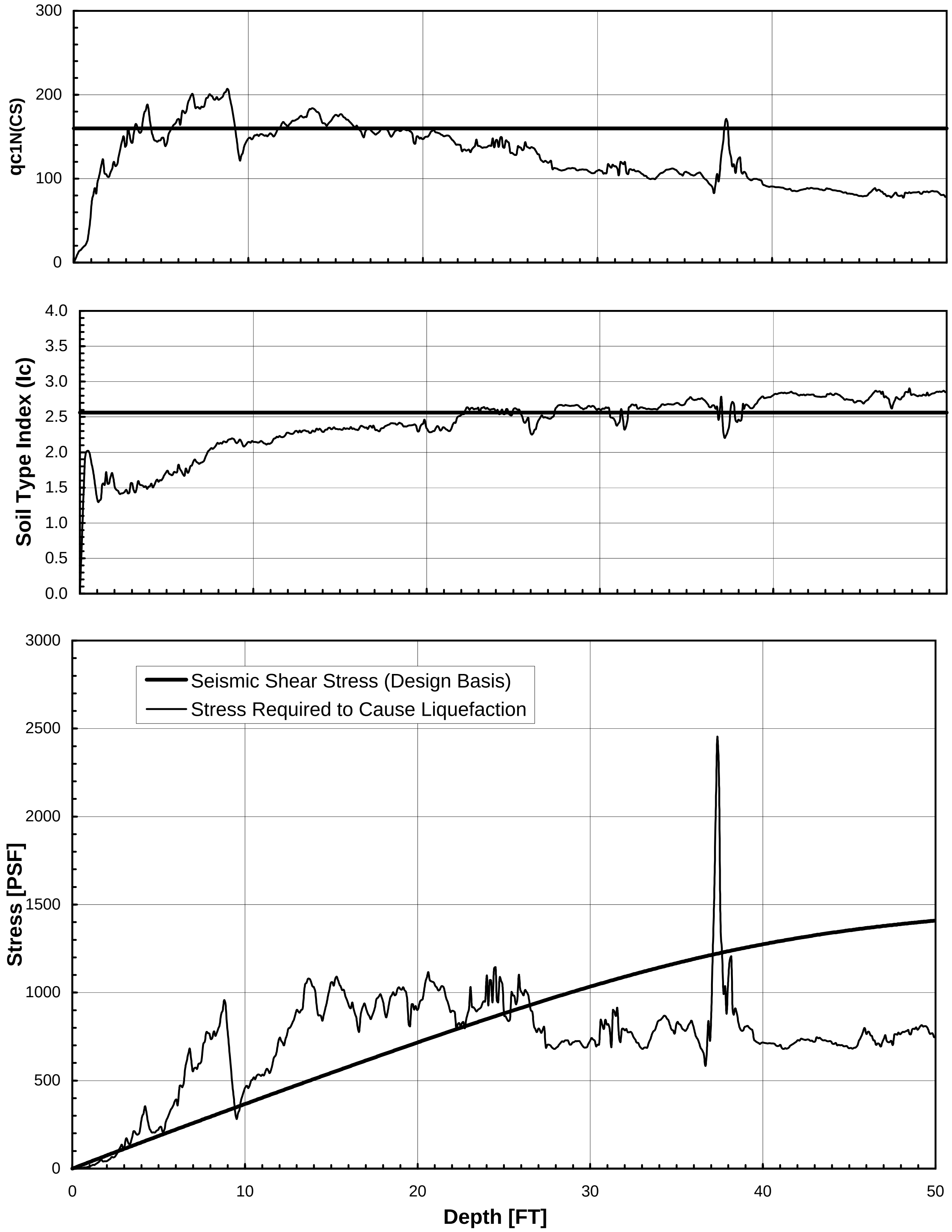
SETTLEMENT ANALYSIS (CPT-14)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-5.4



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LIQUEFACTION ANALYSIS (CPT-15)

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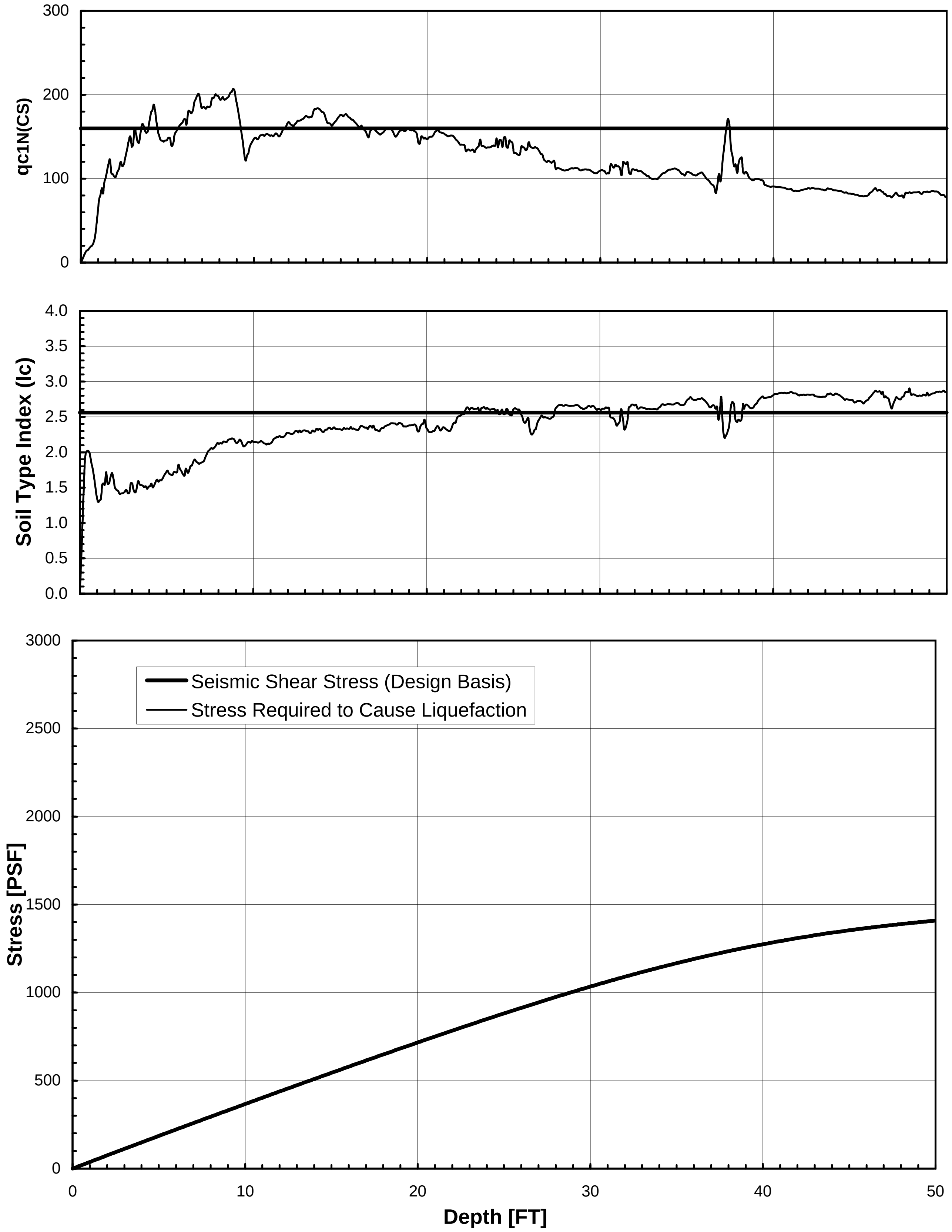
Document No. 06-0015
FIGURE F-6.1



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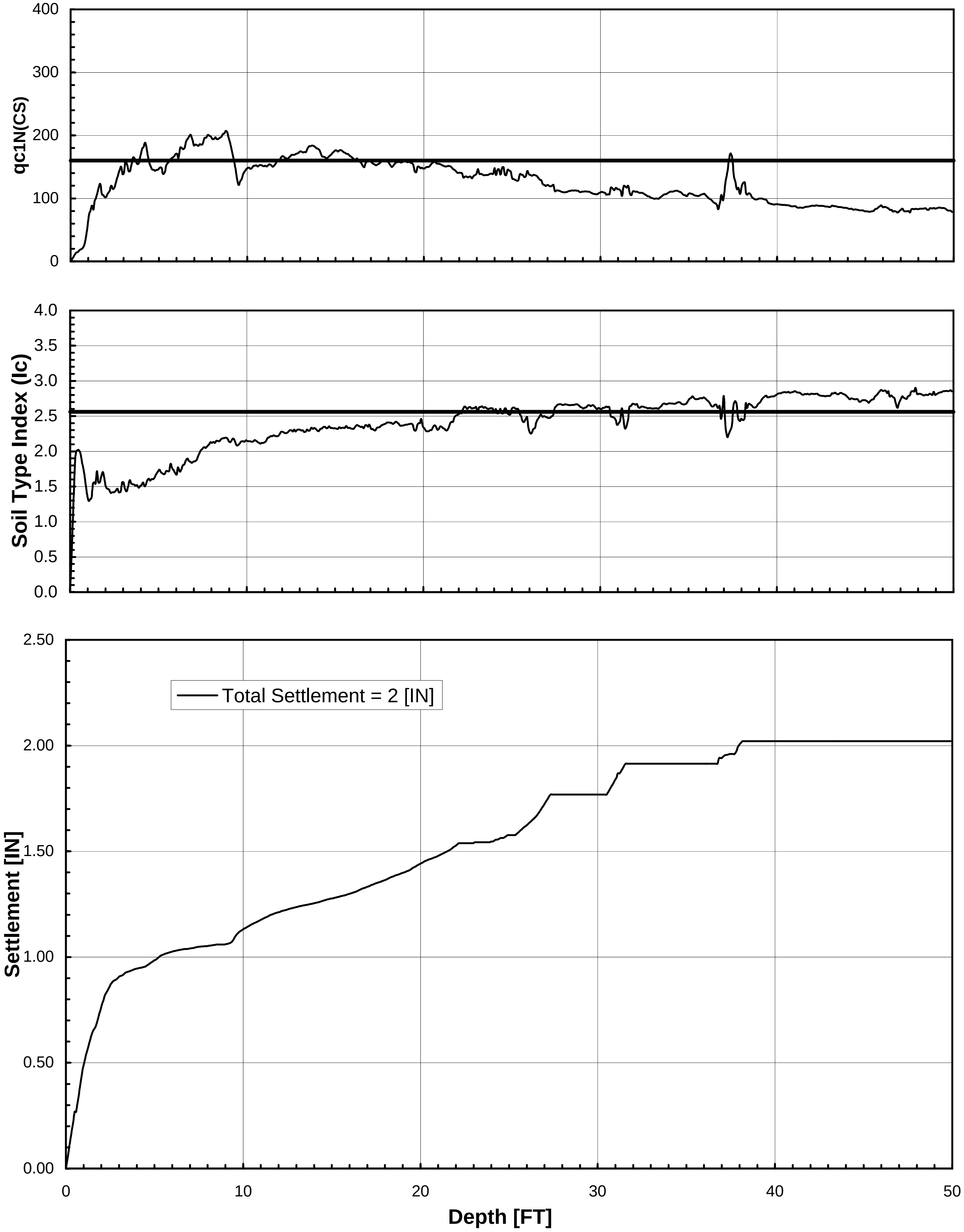
Document No. 06-0015
FIGURE F-6.2



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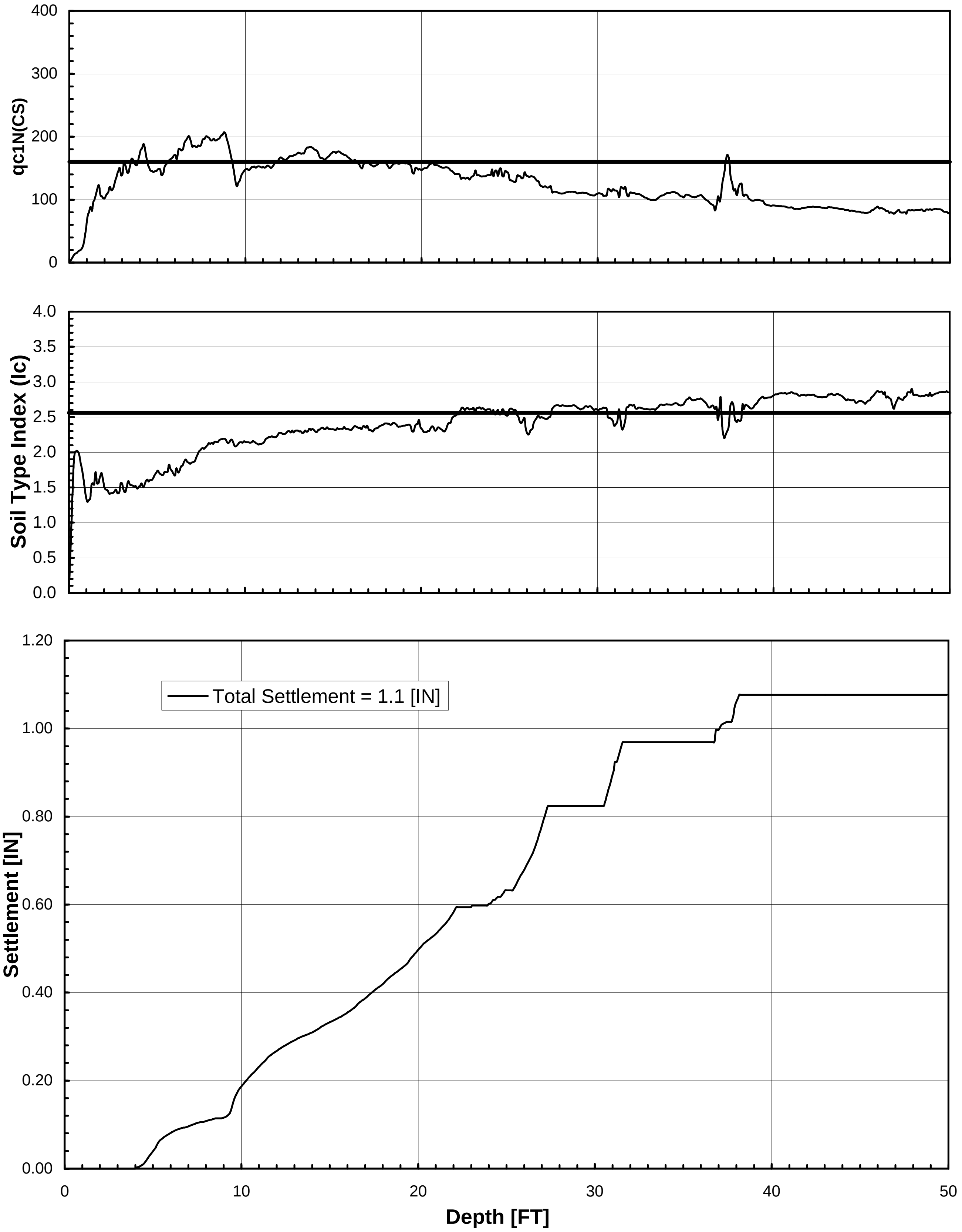
SETTLEMENT ANALYSIS (CPT-15)

Project No. 0554-075-00
Document No. 06-0015
FIGURE F-6.3



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SETTLEMENT ANALYSIS (CPT-15)

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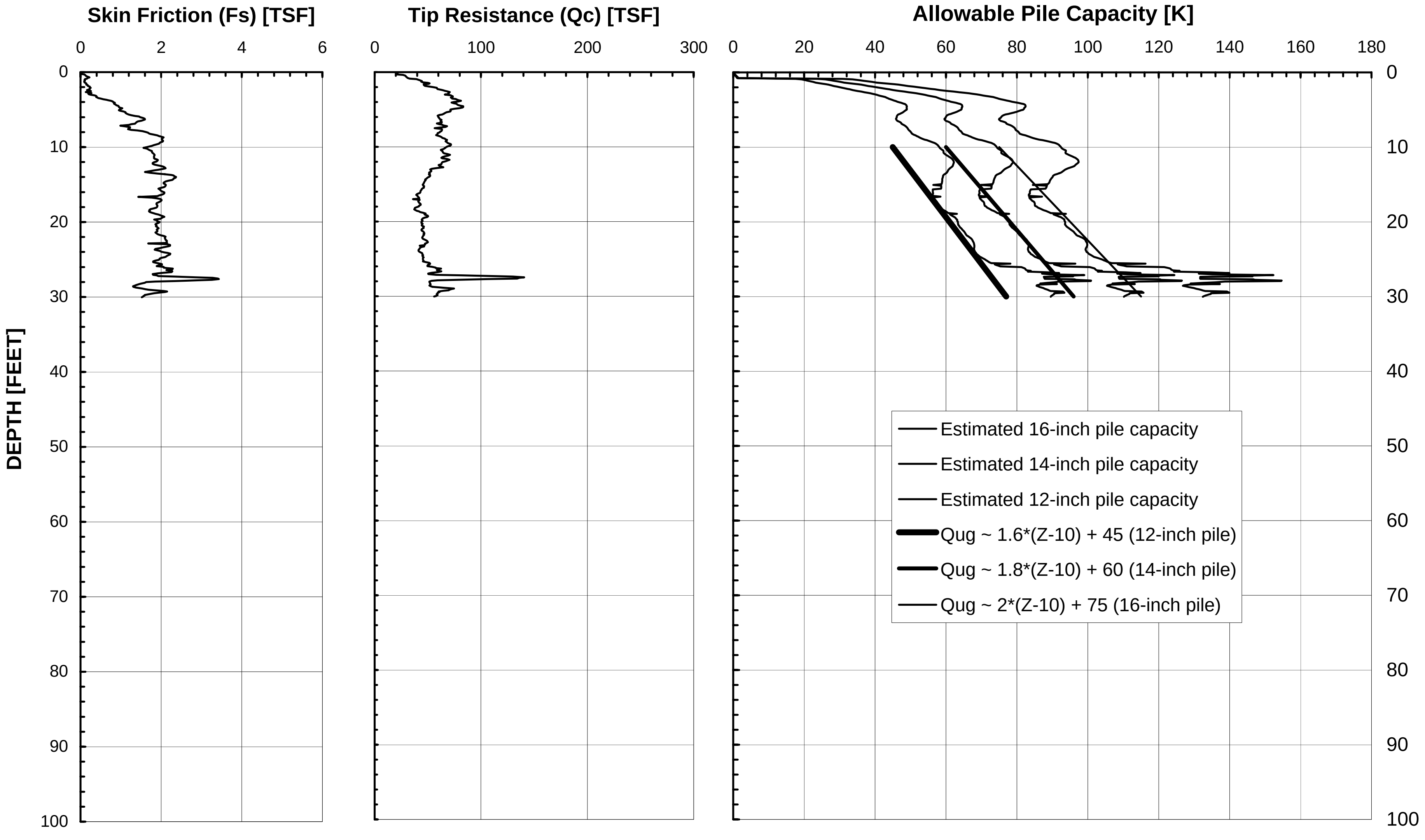
Document No. 06-0015
FIGURE F-6.4

APPENDIX G

PILE ANALYSIS

Pile load capacity analysis was conducted on the data gathered from the CPT soundings using the methods developed by Bustamante and Gianeselli (1982). The analysis assumed that driven, precast, square concrete piles will be used. Pile diameters of 12, 14 and 16-inches were assumed. The results of the CPT pile analyses were combined with conventional analytical techniques to develop the pile recommendations presented in this document. The CPT pile capacity analyses are presented in Figures G-1 through G-6. Note that a factor of safety of 2 is included in the allowable pile capacity estimates presented in these figures.



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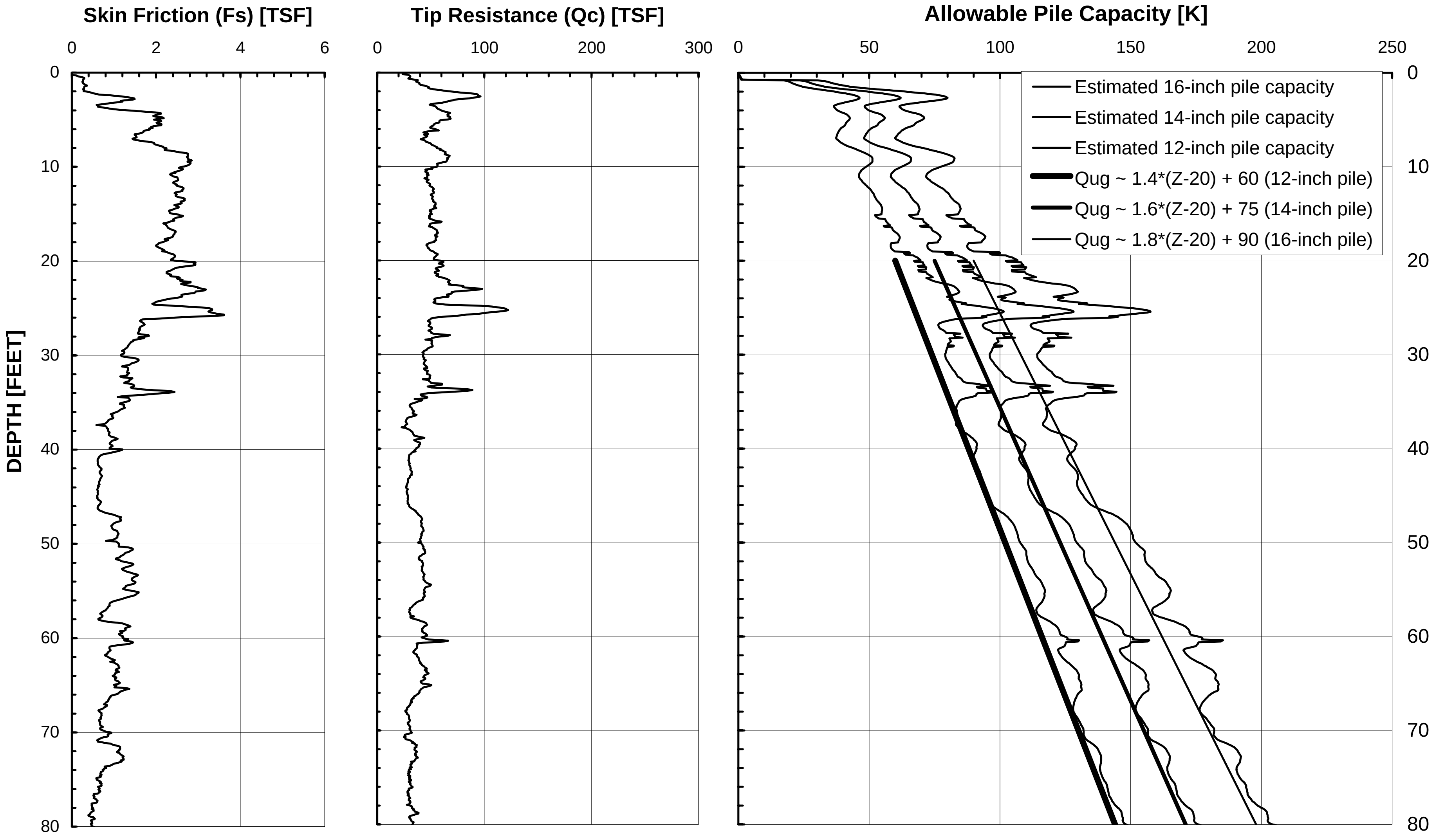
PILE CAPACITY CHART (CPT-7)

Project No. 0554-075-00
Document No. 06-0015
FIGURE G-2



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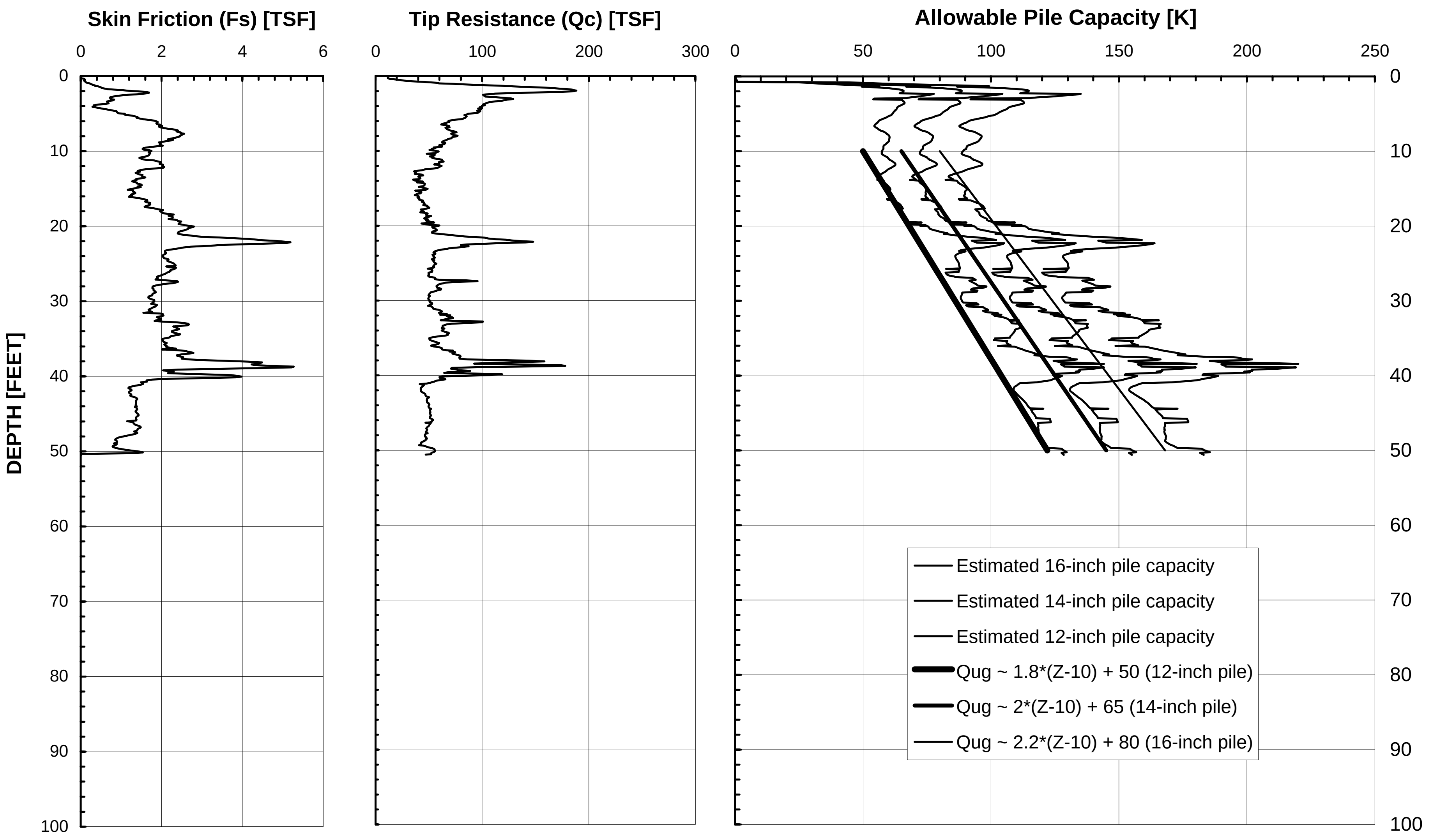
PILE CAPACITY CHART (CPT-10)

Project No. 0554-075-00

Document No. 06-0015

FIGURE G-3



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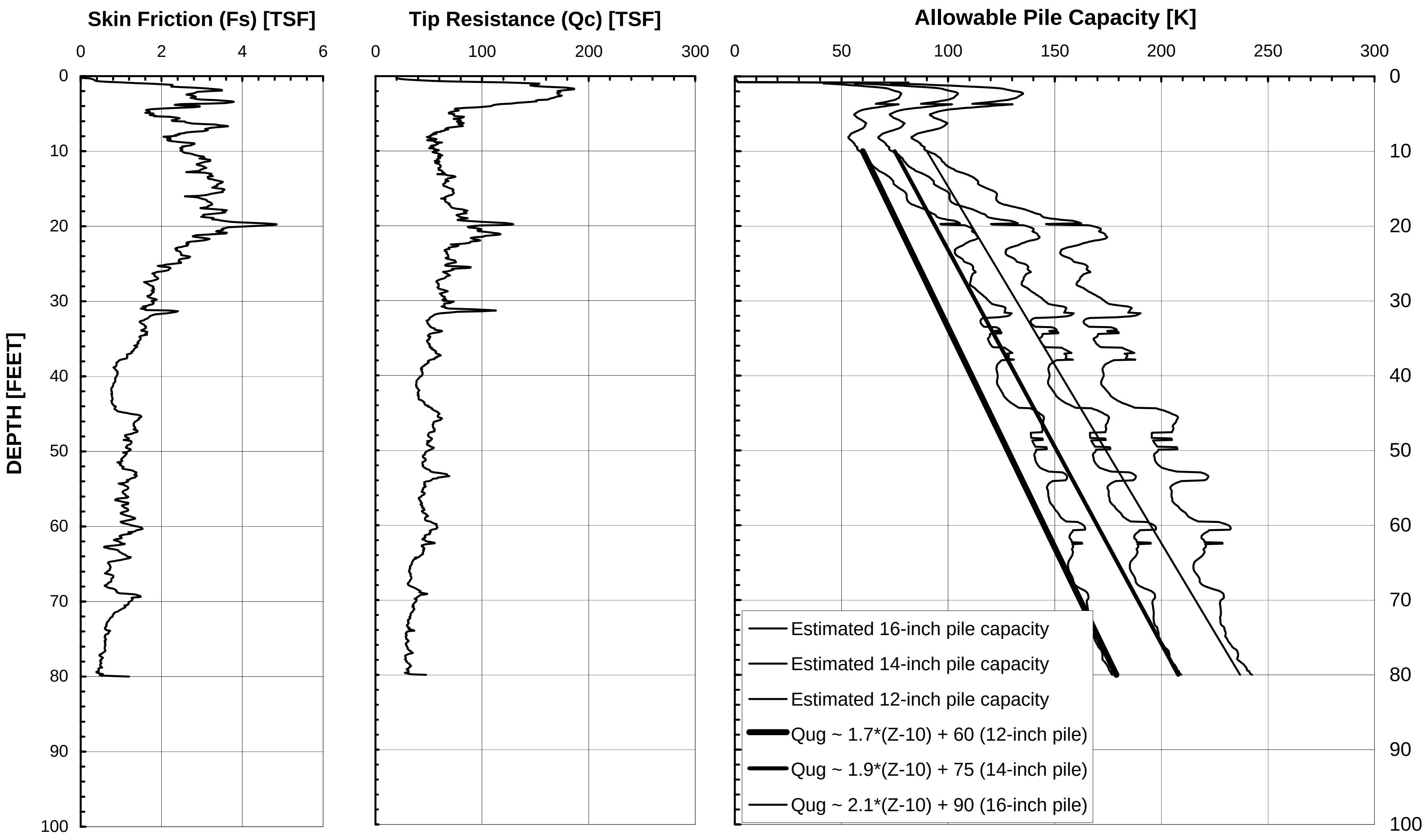
PILE CAPACITY CHART (CPT-11)

Project No. 0554-075-00

Document No. 06-0015

FIGURE G-4



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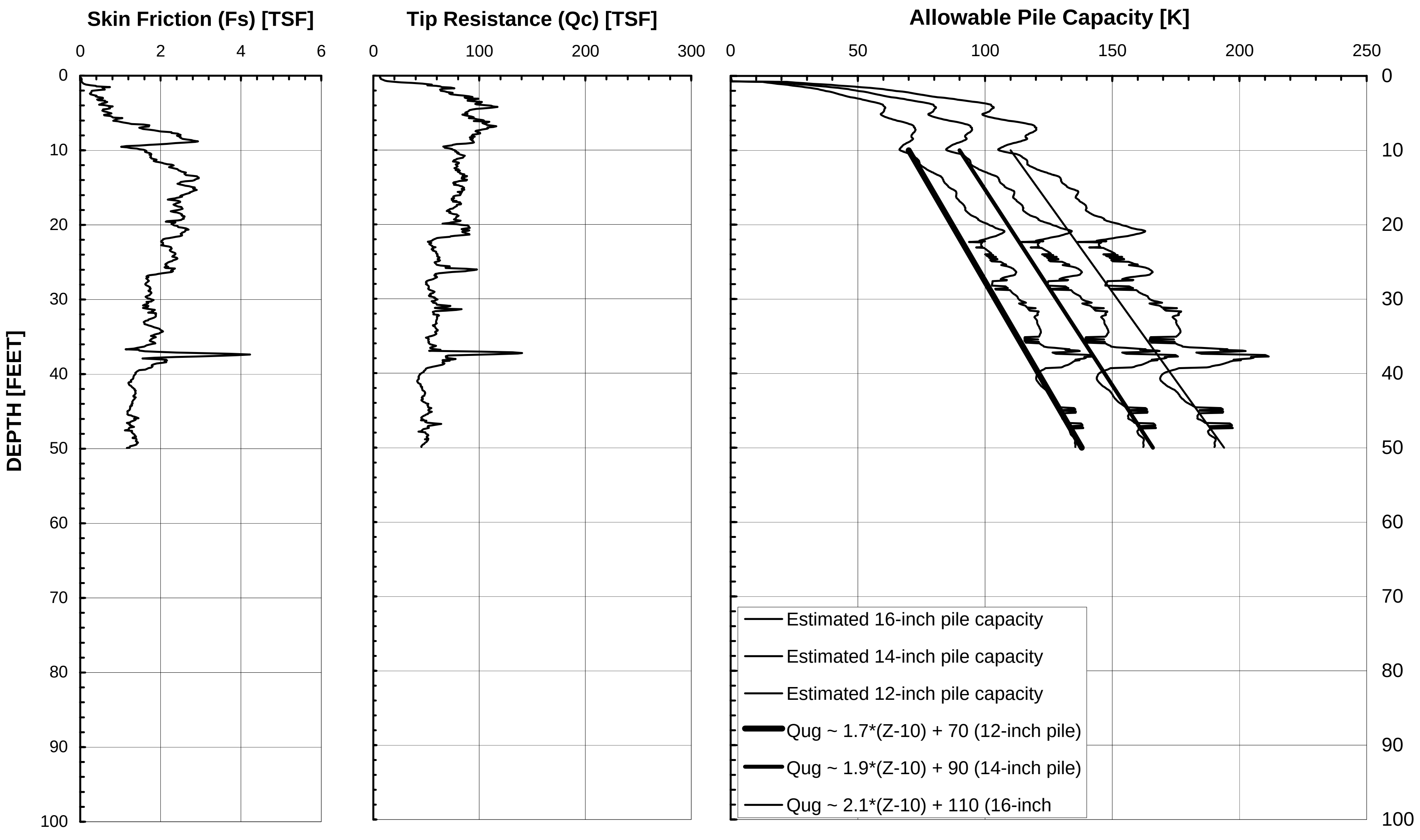
PILE CAPACITY CHART (CPT-14)

Project No. 0554-075-00

Document No. 06-0015

FIGURE G-5



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PILE CAPACITY CHART (CPT-15)

Project No. 0554-075-00

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FIGURE G-6